CIVIL ENGINEERING

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Courtesy Salt River Valley Water Users' Association

Mormon Flat Dam on the Salt River, Near Mesa, Ariz.
Arizona Has Recently Adopted a Building Code for Dams. Refer to Article on Page 200

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APRIL 1933

Among Our Writers

J. A. FRAPS entered the U.S. Indian Irrigation Service in 1925. While there he assisted in the design of the Coolidge Dam. Since 1928 he has been in the State Engineers' Office of Arizona, on the supervision of irrigation districts and dams. He recently supervised the design and construction of a 75-ft. ambursen-type dam near Prescott, Ariz.

JULIUS LASKER, after graduation from Brown University in 1909, took up railroad bridge design, both in concrete and steel. Beginning in 1916, his work involved the structural design of theatres, office buildings, power houses, and a dry-dock. His work in Jerusalem, described in this issue, was completed in 1929.

H. S. HAWORTH is a structural engineer with the Standard Oil Company (Indiana). He has designed and supervised the construction of several reinforced concrete industrial buildings for this concern and a number of others for engineers and contractors in Chicago.

FREDERICK J. CONVERSE has been an instructor in civil engineering at the California Institute of Technology since 1920. For many years he has been actively interested in structural problems, and in foundation and soil investigations. Recently he became associated with the firm of Labarre and Converse, of Los Angeles, Calif.

WILLEM RUDOLFS was born in Holland and educated there as a biological chemist. He is a professor at Rutgers University and Chief of the Sewage Research Division of the New Jersey Agricultural Experiment Station, where biological, chemical, and engineering studies on trickling filters have been conducted since 1921.

NOEL S. CHAMBERLIN is a graduate in chemical engineering of the University of Pennsylvania. For the past three years he has been a graduate assistant in the Division of Sewage Research and an instructor at Rutgers University, devoting his time to chemical studies in sewage treatment.

SEARCY B. SLACK has been engaged in highway and bridge construction almost continuously for 20 years. For the past 13 years he has been Bridge Engineer for the State Highway Board of Georgia. During this time the board's laboratory was established and equipped, under his supervision.

RALPH H. CHAMBERS is an acknowledged expert on heavy construction, difficult foundation, subaqueous and underground work, and hydro-electric power installation. Since 1929 he has been director and Vice-President in charge of engineering of The Foundation Company.

PHILIP H. CORNICK, after wide experience in engineering economics, chiefly in the Southwest and in Mexico, went into governmental research work in 1919, and since that time has been engaged in state and municipal surveys in the fields of property taxes, special assessments, debts, and conservation.

HERMAN H. SMITH has been for 30 years in the service of the City of New York. From 1921 to 1928, he was Deputy Chief Engineer of the Board of Estimate and Apportionment, and since that time has been Chief Engineer. The work of the Department of City Planning has recently been absorbed by this board.

GEORGE H. HERROLD, chairman of the Society's City Planning Division, is Managing Director and Engineer of City Planning, for St. Paul, having been drafted for this work from the Department of Public Works in 1920. Of the 13 major projects in the recent St. Paul bond issue improvement program 11 are City Planning projects.

G. G. MCCAUSTLAND was associated with Black and Veatch in Kansas City from 1917 until 1930, when he became City Plan Engineer of the Kansas City Plan Commission. He is now engaged on the ten-year program of improvements for that city.

OLIVER D. KEESE has been engaged on various phases of public improvement work for many years. Since 1921 he has been with the County Surveyor of Los Angeles, and now has charge of the plans and assessments for the extensive improvements in Los Angeles County.

GEORGE C. D. LENTH entered the engineering service of the City of Chicago in 1905. He became Assistant Engineer and later Assistant Chief Engineer of Sewers of the Board of Local Improvements, which latter position he held until 1923, when he left to become secretary of the Clay Products Association.

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Arizona Adopts a Code for Dams—Part I

A Résumé of the Regulations Adopted by the State Engineer for Their Design and Construction

By J. A. FRAPS
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ENGINEER, STATE ENGINEER'S OFFICE, PHOENIX, ARIZ.

ON October 10, 1932, a Code on Dams was adopted by the State Engineer of Arizona as an official publication. A tentative code had previously been submitted to a number of hydraulic engineers and to officials in other states having laws pertaining to the supervision of dams. The valuable criticisms received from these sources were utilized in the preparation of the final code. This résumé of the code as officially published is in two parts, of which the second will appear in the May issue. Comments and criticisms are requested from interested engineers everywhere, particularly from those engaged in state supervision of dams, or in the design and construction of such structures.

IN the Code on Dams, recently issued by the State Engineer's Office of Arizona, foundations, spillways, and outlet works receive detailed treatment, while the design and construction of earth, rock-fill, gravity, vertical arch, and multiple-arch dams are discussed in a general way. This was done because of the belief that careful treatment of foundations, spillways, and outlet works would result in a marked decrease in the number of failures of dams. A review of past failures shows that structural deficiencies are seldom responsible for the loss of a dam, but poor foundations, inadequate spillways, and faulty outlet works have been the direct cause of many total and partial failures. The desirable qualities for concrete in hydraulic structures are discussed, and general specification requirements are given for concrete construction.

There are two important points that should be borne in mind by any one who attempts to adopt Arizona's code in its entirety. First, it was prepared for use in Arizona, and some features are applicable to that region and to no other. Second, the code is not expected to be irrevocable, for many features of design and construction that are uncertain at present will be clarified in the future.

Although it is designated as a "code" on dams, this term does not carry the usual legal meaning. The code is merely a set of rules and regulations for the guidance of the designer and builder. Points at issue between the State Engineer's Office and the owner of a dam will always be settled on the merits of the question, which may or may not be in conformity with the code.

SOUND DESIGN AND ADEQUATE CONSTRUCTION

The purpose of the code is to compile sound, pertinent data relative to the design and construction of dams, because it is believed that such procedure will eventually result in uniform safety and stability. Certain theories of design have become obsolete in recent years, and have been superseded by more exact ones. The use of the old theories can be definitely thwarted by the issuance of a code that states what theories of design are considered adequate. Construction methods have also been undergoing a distinct change, but it does not necessarily follow that all the changes have been beneficial in the case of hydraulic structures. The code can exclude construction methods that are known to result in a structure that differs widely from the original design.

New designs may depart materially from those discussed in the code, and they are not discouraged. Any new design will receive the same treatment by the State Engineer's Office as one that follows closely the stipulations of the code. New designs, based on sound engineering principles and resulting in more economical structures, are needed for the development of irrigation and power projects. Designs of this nature must necessarily be accompanied by detailed information. The use of unit stresses higher than those stipulated in the code is discouraged, but will be considered if sufficient supporting data are supplied.

Several types of dams are not discussed in the code. The omission of any particular type does not indicate disapproval of it, but rather that it has been adequately covered in a discussion of some other type.

GOOD FOUNDATIONS ESSENTIAL

The State Engineer's Office considers that the most essential feature for a good dam is an adequate foundation. Great care will be exercised, not only in the preliminary explorations, but also in the treatment of foundations during construction. It is realized in this connection that geologic conditions are often too complicated for the engineer to comprehend fully, and when this situation arises, an expert geologist will be called into consultation.

There are two reasons for making detailed foundation explorations before construction is started. As far as the State Engineer's Office is concerned, it is only necesssry to know whether the rock is suitable for the type of dam proposed. The owner, however, needs more detailed data on foundation conditions to secure a reliable estimate of cost.

The extent to which foundation explorations should

be made depends on the type and height of the dam. Prior to the construction of a small dam, a casual inspection will be all that is needed in many cases. If a high dam is to be constructed, very careful and thorough foundation explorations are necessary. As a rule, diamond drillings are essential, and in some cases tunnels and shafts are required before a decision can be reached. Wash borings are generally inadequate, because with them it is impossible to distinguish between foundation rock and a large boulder.

Courtesy Salt River Valley Water Users' Association

HORSE MESA DAM ON THE SALT RIVER, NEAR PHOENIX, ARIZ.
Preparing the Foundation Rock in the Stream Bed for the Concrete

Geologic faults are considered a menace to dams in Arizona, since slight earthquake shocks have been known to occur in nearly all parts of the state. Conditions involving faults demand the services of a competent geologist, and would not be passed on by the personnel of the State Engineer's Office. Where there is evidence of faulting it is probable that greater preliminary explorations than usual will be required.

A number of general requirements for foundations are included in the code. If the dam is to be placed on rock, the foundation material must have the proper degree of imperviousness. The rock must not undergo any material change when it becomes wet, and the foundation materials must have adequate resistance against weathering. If the question of strength is of importance, it is prescribed that tests be made to determine the ultimate compressive strength of samples of rock. This is not a serious problem on the majority of dam sites in Arizona, but must be solved for all extremely high dams. In some cases, the rock may be sufficiently hard but may have been shattered to such an extent as to make it undesirable for the foundation of a dam.

It is prescribed that thoroughness and care be exercised in the preparation of foundations. No effort will be spared by the State Engineer's Office to obtain the type of foundation on which the design was originally based. Unexpected foundation conditions may require that the design be revised, or in extreme cases, that a new type of dam be adopted.

Foundations for earth dams may be of either earth or rock, but provisions must be made for proper treatment of the material. If the dam is placed on earth, all organic matter must be removed, and the foundation material must be treated to obtain a good bond with the new fill. Rock foundations require core walls from the foundation up into the body of the dam, or a number of cut-off trenches carried down into the rock foundation. In high earth dams, where an impervious core wall is used, the foundation below the bottom of the core trench must be grouted.

Generally speaking, it is believed that a rock-fill dam requires a foundation of rock or compact gravel. It is necessary to remove all organic matter and loose, unstable material from the site. The impervious cut-off wall common to this type of dam should be embedded in firm, sound rock, and the foundation below the bottom of the core trench should be grouted.

It is not considered desirable to place a concrete dam, other than the floating-weir or low buttressed types, on any foundation material other than rock. In the preparation of a rock foundation, all the overburden must be removed from the stream bed and the abutments to secure sound, unweathered rock. Large blasts are seen as a possible source of danger to foundation rock. All loose rocks must be barred or wedged from the foundations to ensure a better footing. There must be little, if any, downward slope of the foundation from heel to toe. Particular care must be taken to prepare horizontal footings for the abutment sections of curved gravity dams. Care must also be taken to make the surface of the rock foundations rough in order to resist sliding. Rock abutments for arch dams must present a surface at right angles to the arch thrust.

When steps are made for any type of dam, they must be low in comparison with the height of the structure. A high step is considered a point of weakness, and is believed responsible for the cracking of concrete buttresses in some cases. It is also considered poor practice to permit the surface of the rock foundation to rise from both sides to a point in the shape of a blunt wedge.

After final barring and wedging have been completed, all rock surfaces are to be cleaned with water under high pressure. When this has been done, all water and foreign matter are to be removed from depressions in the rock surface. Before placing concrete, a grout consisting of one part of cement and one and one-half parts of sand must be deposited and spread to assist in obtaining a good bond between the concrete and the rock. Special care must be taken to grout springs or to transmit the water from them through drains before the concrete is placed.

The foundations for a gravity or arch dam must be grouted along the entire upstream face. It is not possible to prescribe definite spacing or depth for grout

holes, or definite grout pressure, since these factors depend on the height of the dam and the character of the rock. Generally speaking, grouting details will not be approved until the foundation material is fully exposed. When excellent rock is available, grouting may be done before the concrete is placed by sealing the grout pipes at the surface. It is considered preferable, however, to grout the foundations after water has had an opportunity to reach all the seams and cracks under the dam.

SPILLWAYS OF VITAL IMPORTANCE

On account of the long list of dams that have failed as a result of inadequate spillways, the code considers that the problem of spillways is of vital importance. Records of flow are not available over long periods of time for the majority of streams in Arizona. Therefore a factor of uncertainty is introduced in the determination of the maximum probable flood. When records of stream flow are meager, a liberal allowance is to be made for spillway capacity, particularly in the case of earth and rock-fill dams.

Spillway channels must offer no danger to the dam proper. This is more important in the case of an earth-fill dam than of a concrete dam founded on good rock. The most favorable location for a spillway is through a saddle or other depression well removed from the dam, but such a location can seldom be obtained. Spillways may be placed adjacent to earth-fill dams, if proper means are taken to protect the fill from the water in the spillway. It is not considered safe to place the spillway on top of an earth embankment.

A common and acceptable spillway for concrete dams consists of side spillway channels. Unless the rock is of exceptionally good quality, the floor of the channel is to be paved with concrete. If necessary, spillway retaining walls should be designed to include dynamic as well as static forces. When water is carried directly over the top of a concrete dam, a special problem is involved. The design must then include means to prevent damage to the dam from vibration and scour at the toe. Tunnels may be employed to carry flood flows around a dam. When they are used, care must be exercised not to disturb the surrounding rock during excavation. Tunnels of this nature are to be lined with concrete, and pressure grouted.

When stream-flow records for a long period of time are available, the maximum probable flood can be obtained from frequency curves. Characteristics may be secured from a study of hydrographs of recorded floods. With this information, a spillway can be designed to pass with safety the required volume of water. The required spillway capacity may be reduced by the effect of reservoir storage above the spillway lip. If the records of stream flow are meager, determination of the maximum probable flood must depend largely on the application of personal judgment. In cases of this kind, the State Engineer's Office must necessarily be more conservative than otherwise. Solutions may be obtained from a comparison with adjacent or similar watersheds. An accurate relation between rainfall and run-off is considered difficult to obtain and dangerous to apply.

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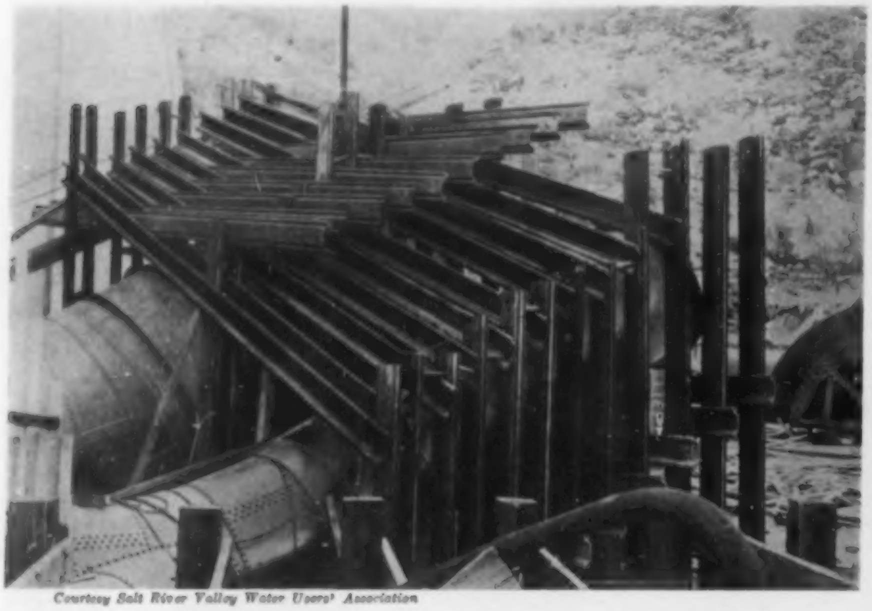
COOLIDGE DAM ON THE GILA RIVER, NEAR GLOBE, ARIZ.
Multiple-Dome Type with Two Outlet Towers; Automatic Spillway Gates Not in Place When Picture Was Taken

If the spillway is of a common type, the discharge may be computed by means of generally accepted formulas. Consideration must be given to the shape of the spillway lip, the slope of the channel downstream and upstream from the crest, and obstructions such as piers. The formula, $Q = CLH^{1.5}$, is acceptable for a wide variety of spillway conditions. Corrections must necessarily be made for entrance velocity and end contractions.

Conditions may be such that the ordinary type of spillway cannot be utilized. In this case, it is recommended that tests on models be used to obtain the desired cross section at the spillway crest and at different points in the spillway channel. Tests on models will be required in cases where a mathematical solution is subject to great error.

Freeboard, as defined by the code, is the vertical distance from the reservoir surface, during maximum spillway discharge, to the crest of the dam. The purposes of an ample freeboard are to guard the dam against overtopping from waves, and to serve as an additional factor of safety in the event of a flood greater than the assumed maximum. Obviously, freeboard can be of no assistance in providing additional spillway capacity if heavy winds demand the full allowance for wave action.

A greater freeboard will be required for earth and rock-fill dams than for concrete structures, since the former cannot be overtopped even to a slight extent without danger. A minimum freeboard of 3 ft has been prescribed for earth and rock-fill dams. The code recommends that the proper freeboard allowance for earth and rock-fill dams shall be computed from the formula, $H = 1.5\,(1.5\,d^{0.5} + 2.5 - d^{0.25})$, where H is the vertical height of freeboard in feet, and d is the distance in miles from the dam to the most remote point along the shore of the reservoir, measured across open water. A vertical concrete parapet wall may extend above the body of an earth or rock-fill dam to provide freeboard.



Courtesy Salt River Valley Water Users' Association

STEWART MOUNTAIN DAM ON THE SALT RIVER, NEAR PHOENIX, ARIZ.
Heavy Reinforcing Around Outlet Pipes

Any concrete dam founded on poor rock is to be subject to the same requirements regarding freeboard as an earth or rock-fill dam. A concrete dam founded on good rock must have a freeboard equal to two-thirds the amount prescribed for earth and rock-fill dams.

OUTLET WORKS—A VULNERABLE FEATURE

Spillway gates that depend on hand operation by a single attendant will not be permitted if the failure of the attendant to operate the gates might endanger the safety of the dam. If two or more attendants are constantly on duty, hand operation will be allowed. Hand operation must be rapid enough to raise or lower the gates within the time limit imposed for the safety of the dam. Automatic gates are required to have an auxiliary mode of operation. There should be enough space between spillway piers to permit the passage of floating debris, or suitable means should be taken to prevent such materials from entering the spillway channel.

Stringent requirements are considered necessary for the design and construction of outlet works on earth and rock-fill dams. The problem is not so serious for concrete dams, but it is required that certain principles shall not be violated.

For the location of an outlet pipe in earth or rock-fill dams, the State Engineer's Office favors a trench or tunnel in rock situated in one of the abutments. If a trench is excavated through solid rock, the pipe may be surrounded completely with concrete or may be supported at intervals along the trench. In either case, it is specified that any continuous, smooth surface shall be broken by means of collars or vertical walls. When a pipe passes directly through the body of an earth or rock-fill dam, special precautions must be taken to prevent the seepage of water along the smooth outer surface of the pipe, and to prevent unequal settlement along the pipe. Seepage may be counteracted by means of collars around the pipe. Particular care in back-filling and in the preparation of the footing for the pipe will prevent unequal settlement.

Tunnels are often well adapted for reservoir outlets. As a general rule, it is necessary to line the rock with concrete or steel. When hydrostatic pressures are extremely high, it will be necessary to use steel lining plates. Concrete dams of the gravity, vertical-arch, or buttress type may have outlet pipes that pass through the body of the main structure. There is danger, of course, when a large pipe passes through a thin arch of short span.

An outlet pipe must be designed to withstand all pressures due to water, as well as to the material above the pipe. It is desirable to have a greater thickness of pipe than is needed to carry stresses safely, in order to permit some deterioration without overstressing. Unless a large number of outlet pipes are used to regulate flood flows, the State Engineer's Office is not concerned with the capacity of such pipes.

It is usually required that an earth or rock-fill dam have a regulatory gate or valve at the upstream end of an outlet pipe. Permission will be granted to place the gate or valve at the downstream end of the outlet pipe, provided the pressure pipe is placed within a larger pipe in such manner that any leaks would discharge into, and drain through, the annular space between the two pipes. It is prescribed that the operation of gates and valves shall be strictly safe and that the mechanism shall be reasonably permanent.

The failure of any structure at the upstream end of an outlet pipe might jeopardize the safety of the dam. For this reason, careful supervision over intake towers must be exercised. They must be designed to withstand not only direct compressive stresses, but also any bending stresses that may come on them. Trash racks become of interest to this office when outlet pipes are used to assist in the regulation of flood flow. In this case, the devices used to remove trash must be designed to operate at all times, regardless of the height of water in the reservoir.

REQUIREMENTS FOR EARTH DAMS

The minimum crest width for any earth dam has been placed at 10 ft. Other specified crest widths for various heights of dam are given in Table I.

TABLE I. MINIMUM CREST WIDTHS FOR EARTH DAMS

HEIGHT	CREST WIDTH
15 to 40 ft	10 ft
100 ft	15 ft
150 ft	20 ft

The recommended crest widths for dams of intermediate height are proportionate to the minima listed in the table. It must be realized that they are subject

to change because they apply to good materials and construction. Minimum side slopes adopted for earth dams are shown in Table II.

Table II. Minimum Slopes for Earth Dams

Height	Upstream Slope	Downstream Slope
15 to 40 ft	2:1	$1\frac{1}{2}$:1
40 to 100 ft	$2\frac{1}{2}$:1	2:1
100 to 150 ft	3:1	$2\frac{1}{2}$:1

Height as used in this table is measured from the lowest point in the original stream bed to the crest of the dam. The side slopes are for dams constructed of good materials under rigid specifications. Inferior materials or specifications will require flatter slopes.

The State Engineer's Office considers that an impervious core is desirable for earth dams that exceed 100 ft in height. The core may be of concrete or may be formed by hydraulic methods. When an impervious core is not used, the dam must be constructed of selected materials of suitable quality. A vertical reinforced-concrete wall may be used to provide an impervious core. The best locations are considered to be along the upstream edge of the crest of the dam or slightly farther upstream. Thicknesses and reinforcing depend on the height of the dam. If selected materials are placed near the center of the dam by the hydraulic method, a thickness of core equal to the vertical distance below the crest is recommended. Construction by the hydraulic or semi-hydraulic fill method is believed to result in excellent structures.

The code specifies that means shall be taken to prevent erosion of the downstream face. A facing of gravel or a system of berms and drain ditches will be required. It also prescribes that any part of the downstream face which is subject to erosion from the swirling action of the water discharged through outlet works or spillways shall be adequately protected. Random rock riprap is considered a suitable means of protection against wave action on the upstream face. A concrete slab may also be employed, but waves will run farther up a smooth slab of this kind than they will on random rock riprap. If a concrete face is used, it will not be considered as satisfying the additional requirement for an impervious core.

SPECIFICATIONS FOR DRAINS

Drains are usually considered necessary for earth-fill dams. The sole purpose of drains, as viewed by the code, is to carry water away from the toe of the dam, and thus prevent undue settlement that might result from waterlogging. If drains are placed too far back in the fill, they increase the slope of the hydraulic gradient and thus violate the assumptions used in the design. Open drains are deemed preferable to those formed of crushed rock or similar materials.

An earth dam may be composed of any class of material if the slopes are made flat enough. Economy will usually demand the selection of good materials. All the earth that goes into a dam must meet the specification requirements for the design adopted. Frozen earth is not to be placed in a fill, and no material is to be placed on a frozen surface. When a dam is constructed by sprinkling and rolling, the degree of imperviousness required for the cross section of the dam must be obtained. When rock is used at the downstream toe of a dam in order to assist in drainage, the material is to be of a durable and weather-resisting character. Materials going into a hydraulic or semi-hydraulic earth fill must be carefully controlled. Detailed rules are not laid down for the control of materials or for tests during construction, but such rules will be worked out during the progress of the work.

When an earth dam is constructed by sprinkling and rolling, the layers are not to exceed 6 in. in thickness when compacted. It is required that the earth shall be moistened to such an extent that maximum compactness

Courtesy Vic H. Householder

Gillespie Multiple-Arch Dam, Gila River, Arizona

Overflow Structure with Stilling Basin on the Apron. Sluice Gates in Foreground

will be secured. Care must be exercised to secure a good bond between the fill and original earth, and between successive layers in the fill. The roller is required to weigh not less than 2,000 lb per lin ft of tread, and to pass over each part of the fill at least three times.

When constructing hydraulic-fill dams, there must be sufficient control over the materials going into the structure to ensure the required width and composition of the inner core. Construction methods must be such as to prevent runs of coarse materials from extending into the core. The rate of construction will depend on the rapidity with which the core is consolidated and dried out.

The dam must be constructed to a height sufficiently above the finished grade line to compensate for settlement after the construction is completed. The reservoir must be filled as slowly as possible, and the dam must be watched carefully during the first filling.

Nearly all streams in Arizona are subject to high floods twice each year. It is impossible to construct any earth dam, except minor structures, during the dry period. Adequate means must therefore be provided to care for floods during construction. These facilities are to be designed by the owner of the dam, and are subject to the approval of the State Engineer's Office. Generally speaking, it is required that the maximum probable flood be accommodated through by-pass channels. Water may be stored in the reservoir during construction, but must never be closer than 30 ft from the crest of the dam.

[*To be concluded in the May issue*]

Engineering Experiences in the Holy Land

Problems of Up-to-Date Construction Under Ancient Handicaps

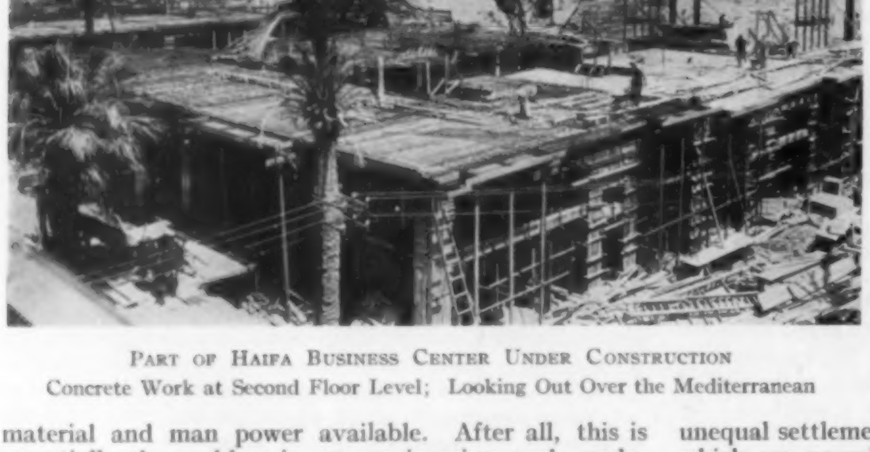
By Julius Lasker

Structural Engineer, Boston, Mass.

ACCOUNTS of engineering work in foreign lands are frequently surrounded with a certain glamor. The working conditions seem unusual; the materials are not always modern; and the local customs and precedents constitute handicaps that appear well-nigh insuperable. All these variable factors apply with special force to modern—yet still old—Palestine. They were particularly disturbing in the two recent jobs described here in interesting detail by Mr. Lasker. The earthquake hazard as applied to the Judean country is perhaps not so well recognized as in other regions. Yet, as this article shows, neglect of this obvious danger, especially considering the stone-and-mud wall construction so prevalent in Palestine, is fraught with grave peril and almost sure disaster. That satisfactory construction as measured by modern standards was attained reflects much credit on the resourcefulness and training of the American engineers.

TO attempt modern building in present-day Palestine is to contend with antiquated methods of labor and construction—to face the necessity of accommodating the old to the modern. Although my observation was confined to a period of only five years, yet some of my experiences should be of interest to American engineers. The two enterprises involved were perhaps among the largest undertaken in recent years in that historic country. The first had to do with a modern business development in the busy port of Haifa. Just as this was about completed, in the summer of 1927, a disastrous earthquake occurred. It was particularly destructive to the buildings of the Hebrew University in Jerusalem, and for the next two and a half years I was engaged in the repair of its old structures and in the building of its new ones.

Although essentially different in their construction and purposes, these two projects had this in common, that they required adjustments of method to meet the exigencies developed as the work progressed, due to the necessity of getting results with the limited resources of material and man power available. After all, this is essentially the problem in any engineering work, under whatever auspices, or in whatever country it is undertaken.



Part of Haifa Business Center Under Construction
Concrete Work at Second Floor Level; Looking Out Over the Mediterranean

Within a stone's throw of the squalid, crooked alleys of the Arab "sook," or market place, of Haifa, may be found the newly built Business Center. It consists of six groups of modern buildings, and has wide, electrically lighted, paved streets, clean granolithic sidewalks, and its own water-supply and drainage system. The Western type of construction was used but was softened by the taste of the East, as expressed in a considerable number of balconies on the upper stories, by groups of stately palms, which escaped destruction when the buildings were laid out, by a fountain spraying its waters in the center of the busy square—all lending to the buildings, about forty in number, a unique and pleasing appearance. The Business Center is situated on the very shore of the bay, so that only the railroad tracks intervene between the buildings and the blue waters of the Mediterranean.

A COOPERATIVE ENTERPRISE

The Haifa Business Center was built as one cooperative enterprise, financed by Mr. Max Shoolman of Boston, Mass., who enabled a group of local business men to erect these buildings on land that they had bought several years previously. Approaching the port by steamer, one can see from a distance these three-story buildings and the central street, which widens out at the shore into a semi-octagon, with a fountain in the middle of a small fenced-in park. Mount Carmel rises immediately beyond, its slopes generously dotted with olive groves and cypresses and the red-tiled roofs of numerous dwellings.

Preliminary investigation revealed that the ground surface of the site was soft and had a fall of about 5 per cent toward the sea. Since fairly hard rock was encountered at a depth of from 7 to 8 ft, it proved economical to found the columns on rock and to carry the walls of the lower story on reinforced concrete girders framed into the columns about 18 in. below the floor level. This arrangement brought the entire weight of the walls on the columns and their foundations, thus eliminating unequal settlement and the resulting cracks in the walls, which are prevalent in Palestinian buildings. A load of 3.5 tons per sq ft was allowed on the foundations.

The stair construction was somewhat novel, being of concrete, precast in special forms. The reinforcement was designed so that only one end of each stair would be supported by the concrete wall into which it was built when the walls were poured, the other end remaining

free. Although the front edge of each step rested on a specially provided narrow seat in the rear edge of the next lower one, each really acted as a cantilever. The remarkable strength of this economical construction was demonstrated several times when heavy steel safes were carried upstairs by four or five men, the entire load being concentrated on two stairs without causing any measurable deflection.

For the first story, which is used for shops, a height of about 18 ft was provided, to allow for balconies on the inside, around the walls, for the office force. The two upper stories, laid out as offices, were all 11 ft 6 in. high, from floor to floor. The extra height is considered necessary because of the subtropical climate. Doors and windows in rooms and in corridors were laid out so as to get cross ventilation, without which buildings in Palestine are unbearably hot in the summer. The cool sea breezes come from the west, and every opportunity was utilized to let them penetrate into the corridors and courts and thence into adjacent rooms, through ventilating shafts, transoms over doors, windows facing each other, and other air passages.

Haifa Business Center Completed and In Use
Buildings and Roadway Are Modern

Structurally these buildings presented no unusual difficulty. However, as engineer in charge of the design and construction, I found myself confronted with some serious problems due to the fact that the country is undeveloped. Chief among them were that caused by the lack of a city water supply in that section of Haifa and that due to the absence of a system of any kind for drainage or sewage disposal.

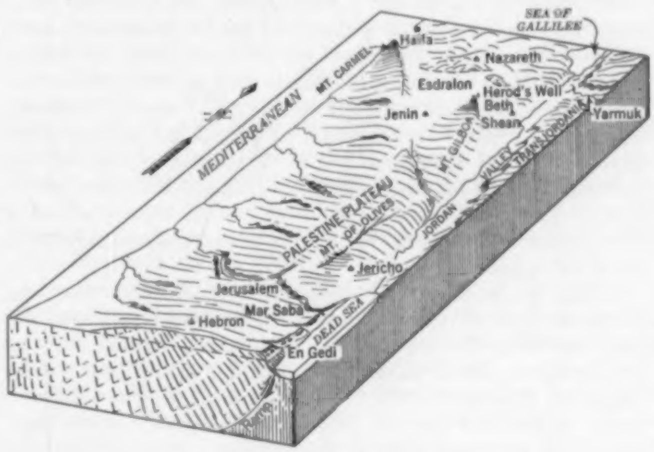
WATER SUPPLY PRESENTS A PROBLEM

For its water supply Haifa depends on cisterns filled by rain water, or on wells dug by individual property owners for themselves. One well, found on the premises, supplied about 50 cu m (13,200 gal) daily after it had been cleaned and deepened. This amount was only a part of that needed for construction purposes, however; hence we dug several more wells, staggering them in two lines nearly parallel to the shore. The depth of these wells, and likewise their yield, varied considerably. In some the water was brackish. In one, which gave only a very small amount of fresh water, a larger pump was installed in the hope of increasing the yield. After a while we did get more water, but it was no longer fresh. Apparently the suction of the pump had overcome the resistance of the sandy layer to the infiltration of the sea water, so that, instead of getting fresh water from the slopes of Mount Carmel, we got a permanent supply of salt water from the Mediterranean. This well had to be abandoned.

When enough yielding wells had finally been developed, they were all connected to one central pumping station. After strengthening the walls and roof of an old building, a temporary water tower was erected on it to serve only during construction. As the buildings were erected, individual tanks were placed on their roofs and kept full by automatic pumps to provide the permanent supply.

When the water of the larger well was examined with a view to using it for a permanent supply, it was found to be badly contaminated and to contain bacilli coli. Investigation showed that some old sewage tunnels from existing buildings in the neighborhood came very close to it. The owners of these buildings were not interested in eliminating the nuisance, and there was no law to compel them to do so. A way was at last found of diverting these tunnels into the general sewerage system of the Business Center. In subsequent examinations, the water showed progressive improvement, until finally it was declared safe for use.

Lack of a sewerage system in Haifa made it necessary to provide an independent installation to take care of the Business Center, with its 100 shops and 400 rooms. It was also essential to provide drainage for the rain water which falls in torrential downpours during the period between November and March. Palestine gets no rain at all during eight months of the year, and most of the annual precipitation of about 30 in. takes place during 20 or 30 days in January and



Sketch Map of Palestine
Surface Relief and Geologic Structure Indicated. Courtesy Bailey Willis and the *Bulletin of the Seismological Society of America*

February. Two separate systems had therefore to be installed, one for sewage and one for rain water. The seashore in front of the Business Center was to serve as a bathing beach and amusement place, overlooked by several cafes, restaurants, hotels, and roof gardens. To

prevent a nuisance being caused by the effluent along the shore line, I designed an anaerobic septic tank of three chambers. The effluent from the last chamber is conducted in a pipe through a culvert under the railroad tracks and into the sea at a distance of about 300 ft from the shore. During the five years that this system

"AUTOMOBILE TRUCKS" OF THE TWENTIETH CENTURY, B.C.
Camel Train Loaded with Stone for Masonry Against the Background of the Partially Erected Library Building

has been in operation there has been no bad odor from it, and the effluent leaving the pipe has been quite clear.

SPECIFICATIONS IN ANCIENT HEBREW

Another problem, unique in engineering annals, was presented by the human material available for the job. This enterprise, the largest in Palestine up to the present time, was carried out entirely by Jewish young men and women pioneers, the "Chalutzim" and "Chalutzoth" who had come to Palestine from various parts of the world. Each group spoke the language of its native land, but no group had any language in common with any other group, except the classical Hebrew, which is becoming in Palestine more and more the language of the shop, of the street, and of the fields. As a result, all the specifications and many of the plans had to be translated into Hebrew to enable the foremen and workers to follow them. But at the time of Isaiah and Micah, reinforced concrete was unknown; nor is there any record of specifications for sheet-metal work or plumbing dating from that period. I therefore found it necessary not only to refresh my boyhood knowledge of Biblical Hebrew, but to do quite a bit of research in order to find equivalents for such modern structural terms as stirrups, diagonal shear, back-vent, and siphon.

Then again, the "Chalutzoth," or women workers, demanded—and were given—the privilege of working together with the men on all parts of the work, such as rod bending, lime slaking, plastering, and tiling. The engineer, therefore, when displeased at times with one thing or another on the job, had to exercise more than the usual self-discipline in the way he expressed his dissatisfaction within range of feminine ears.

As measured by Eastern standards, the work was done so fast—in about 18 months—that groups of Arabs who came to watch operations expressed their belief that "Saitans," or devils, were responsible.

The total cost amounted to about \$650,000. It is interesting to note that labor costs were very low as compared with American standards; a common laborer received the equivalent of \$1.25 to \$1.50 for a day of eight hours, and plasterers and carpenters from \$2.50 to \$2.75 a day. Stone masons, the highest-paid workmen on the job, were given \$3.00 a day.

These buildings were subjected to a real test on July 11, 1927, when a severe earthquake shook Palestine, causing considerable loss of life and extensive damage. They came through this ordeal surprisingly well, far better for example, than the few old buildings that then constituted the Hebrew University on Mount Scopus just outside of Jerusalem. These were severely damaged and were left in imminent danger of collapse. They were of stone, some with walls 4 ft thick, but having been built in the traditional Arab manner, they were veritable death-traps.

Regardless of the fact that from time immemorial earthquakes have devastated certain parts of the country, the Palestinian Arabs persist in building these thick walls, consisting of inside and outside facings of stone with a mud or clay filling in between. Hardly any bond stones or headers are used to tie the facings together. The floor and roof loads, whether carried by stone vaults or by steel beams, are supported on these walls. Hence, when an earthquake shakes a building, the two stone "curtains" of the walls separate like the leaves of a book. The soft fill then gives way under the weight of the floor or roof above, causing it to cave in.

When I visited Nablus (the Biblical Shechem) two days after the earthquake of 1927, I saw scores of bodies being removed from under the debris of houses that had been recently built with these thick walls. In Jerusalem, too, many such houses collapsed, but fortunately with only slight loss of life.

It was imperative that whatever of the university buildings could be salvaged, should be repaired as quickly as possible. Accordingly I was given this interesting assignment. Later a considerable number of new buildings were added to the project.

The first step was immediately to demolish those parts of the old buildings that had been damaged beyond repair. In repairing some of the walls it was necessary to clean out the mud fill between the stones and replace it with concrete, effecting a good bond between the concrete and the stone by means of dowels and bent rods. We also used horizontal rods placed longitudinally at about 3-ft vertical intervals, and other rods, diagonally, at an angle of about 30 deg with the vertical.

INTERIOR OF UNIVERSITY WORKSHOP, SHOWING TYPICAL EARTHQUAKE DAMAGE
Bulging of End Wall, of Mud and Stone Construction, Let Down the Ceiling. Fortunately a Meeting Scheduled to Be Held in This Room Had Been Postponed and Hence No Lives Were Lost

The roof of the Assembly Room, known as Balfour Hall, in the Chemistry Institute, consisted of clay slabs about 3 ft thick, supported on closely spaced parallel I-beams. The slabs and the beams were hidden from view by a suspended ceiling that had been added about three years previously. When parts of this ceiling were removed for the purpose of investigation, the I-beams were found to be so corroded that pieces of the flanges and webs could be broken off by lightly tapping them with a hammer. The interior wall that supported the ends of these I-beams had been so badly shaken that it had to be demolished. Instead of rebuilding it, I decided to carry a central column of reinforced concrete, one story in height, down to the wall underneath, and to reinforce thoroughly a part of this wall so that it would form an adequate base for the column. Two girders, each about 35 ft long, on either side of the central column, were to carry the ends of the new I-beams, between which a reinforced concrete slab was to be poured.

Simple as these proposals sound to an engineer in America, they caused us an untold amount of trouble. Not a single beam or channel of greater depth than 10 in. could be found in Palestine. Those of 10-in. depth or less were narrow-flange German or Belgian sections, which were about 20 per cent weaker than the standard American sections of equal depth. This made it necessary to space these I-beams rather close together. The girders, only 20 in. deep, had to be built up from plates and angles, because ordering rolled sections from England or America would have entailed a delay of about six months. This I knew from sad experience, for it took seven months to get an engineer's level that we ordered from a London supply house of good standing.

Fortunately, we found in Jaffa, about 40 miles distant from Jerusalem, some $^3/_8$-in. plates, each 1 m ($39^1/_2$ in.) wide and 2 m long; and also some angles 4 by 4 by $^3/_8$ in. By cutting the plates longitudinally and splicing them every 6.5 ft, it was found possible to build two girders, each about 20 in. from back to back of angles. Since no adequate punches were available in the small shop where the girders were fabricated, the angles and webs were clamped together and drilled in one operation. Each girder weighed about two tons. Because of the lack of derricks, two 10-in. I-beams, about 60 ft long, were placed flatwise, with one end on the top of the wall and the other on the ground. Then the upper flanges of the inclined beams were greased and along them the girders were skidded up into place.

Lessons Learned from the Earthquake

In the light of the lessons learned from the earthquake, the structural design of the stone buildings was revised. It will be seen from the accompanying photographs that most of the walls cracked along lines running diagonally from the corners or from door and window openings. In a few cases the stones were broken. Usually, however, the crack followed the zigzag line of the joints. Most walls also bulged in or out, and some beams were pulled out of, or pushed into the walls. This indicated that the seismic forces worked not only vertically, but laterally as well.

It was evidently unsafe to allow comparatively thin stone walls to carry roof and floor loads. Hence columns and girders were provided to form a skeleton. The stones, laid in cement mortar, were thoroughly tied in with the concrete backing, but were to function only as a facing for the columns, girders, and walls, which were of reinforced concrete. In addition to the calculated reinforcement, bent rods were used horizontally to tie the corners of the building together. To prevent diagonal cracks, diagonal rods were spaced at reasonable intervals near the ends of the walls. All openings for doors and windows had horizontal and vertical rods around them. The floor slabs were thoroughly tied in with the girders and columns.

Although quarries are plentiful in and around Jerusalem, the provision of a supply of building stone was fraught with difficulties. The plans called for red stones in the front and side walls of the Wolfsohn Library, one of the several new buildings of the Hebrew University. It so happened that the quarries from which the red stone had to be obtained were not on any road, and were not accessible to trucks or even to horse carts. Hence the stones had to be brought one by one to the building site, or to the road, on the backs of mules or camels. If it happened that a stone was too heavy even for a camel, it was transported to the road on a special saddle carried between two camels. There, by means of ropes and an inclined platform, and by dint of a generous amount of profanity, it was loaded on an automobile truck.

One of the Newer University Buildings as Damaged by the Earthquake

Diagonal Cracking of Walls Was Characteristic

Spring Plowing Causes Interruptions

This unique method served well for some weeks, until the time came for spring plowing. Then there was suddenly a shortage of "camelpower." The reason was that the Arab peasant of the twentieth century A.D. still does his plowing just as did his forebears of the twentieth century B.C. He uses a heavy block of wood with a sharp iron tooth fixed rigidly at one end. This tooth is pushed into the ground to the depth of 2 or 3 in. and firmly held down. To the other end of the block is attached a harness which fits any creature capable of exerting a pull, such as a donkey or a camel—or frequently the farmer's wife. During that particular spring, perhaps because of the late rains, the camels had to do more than their share and could not be spared for hauling stones from the quarries. As a result, our entire working schedule was upset for about a month.

With all these handicaps, however, the University buildings, when completed, were creditable both in appearance and strength. Should another earthquake occur in that region, it is believed that they will endure the test satisfactorily.

The architectural plans for the Hebrew University buildings were prepared by Sir Patrick Geddes and Mr. Mears of England. Their Jerusalem representative was B. Chaikin. A retired Chief Engineer of the Eastern Railways of India, L. Green, then a resident of Jerusalem, acted as Honorary Technical Adviser for the University. The writer was in direct charge of the construction.

Reinforced Brick Masonry for Industrial Use

Novel Type of Thin-Wall Construction Proves Economical at Oil Refinery

By H. S. Haworth

Associate Member American Society of Civil Engineers

Structural Engineer, Standard Oil Company (Indiana), Wood River, Ill.

What is believed to be the first industrial building in the United States to be constructed with 4-in. reinforced brick masonry walls was completed by the Standard Oil Company (Indiana) in June 1932, at its Wood River Refinery in Illinois. The details of the method used to reinforce the walls and columns of this interesting building are here described by Mr. Haworth. As a result of his experience at the refinery with other buildings having walls of reinforced concrete, thick plain brick, and corrugated asbestos, he believes that reinforced brick masonry for such construction may be expected to come into general use because of its economy and good appearance. It is also remarkably strong, as shown by the fact that the columns of the compressor building carry a 12-ton crane. The cost data included will be useful for comparison with other types of construction.

Probably reinforced brick masonry was first used by Sir Marc Isambard Brunel in building a tunnel under the Thames River in London. Between February 16 and June 6, 1825, he constructed a reinforced brick caisson 50 ft in diameter and 42 ft high for a shaft at one end of the tunnel. Later a similar caisson was sunk at the other end of the tunnel. It was reported that these caissons were being used in 1931 as part of the London underground tramway system.

The Church of Saint Jean de Montmarte in Paris was designed by M. A. de Baudot and built prior to 1904. Its brick walls, which are 115 ft high, $4^1/_2$ in. thick, and $29^1/_2$ ft between supports, are reinforced with steel wires interlaced through holes in the brick and along the bed joints. Apparently this is the first building to be constructed with thin reinforced brick masonry walls.

More recently, reinforced brickwork has been used quite extensively in India and Japan. Within the past three years, a number of structures using reinforced brick masonry have been built in the United States, among them a serpentine wall in Augusta, Ga. (1930); conveyor supports in Chester, Pa. (1931); sand storage bins and railroad trestle bents, in Wedron, Ill. (1931); a cantilever arch highway bridge, near Philadelphia, Pa. (1931); a water tower enclosure, in Baltimore, Md. (1931); a contractor's office, in Richmond, Va. (1931); and abutments and wing walls for a highway bridge, in Perla, Ark. (1931).

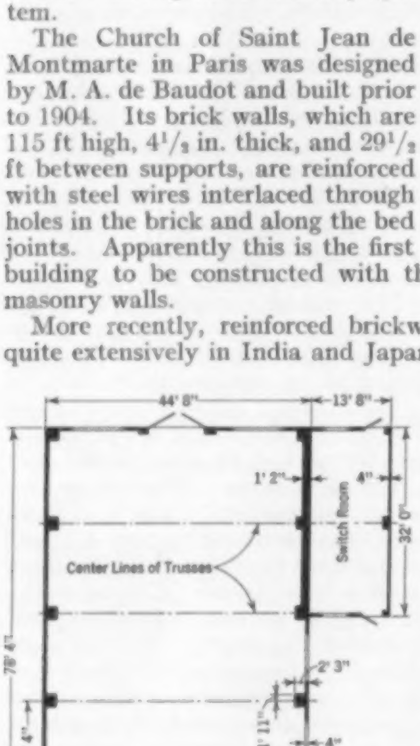


Fig. 1. Plan and Section of Reinforced Brick Compressor House

It is believed that the structure described in this article is the first building with 4-in. reinforced brick masonry walls to be built in this country. This building is located at Wood River, Ill., in the refinery of the Standard Oil Company (Indiana) and is used as a compressor house. The brickwork was started on May 4, 1932, and completed on June 3 of the same year. All the walls and columns of this building are of reinforced brick masonry. The walls, with the exception of two bays, are nominally 4 in. thick. In Fig. 1, the principal dimensions of the structure are given.

The footings under the columns in the main building are of reinforced concrete; those under the side columns, supporting the walls, crane, and roof trusses, are 7 ft by 4 ft by 1 ft 6 in. deep; and those under the door columns, at the ends of the building, are 4 ft square by 1 ft 6 in. deep. The footings under the three columns in the outside wall of the switch room are of reinforced brick masonry and are 4 ft square by approximately 1 ft 8 in. deep. Dowels of the same size and number as the vertical bars in the columns project 2 ft 6 in. above all the footings into the brick columns above.

Construction Materials

All the brick were vertical fiber paving "seconds," having a nominal size of $2^1/_2$ by 4 by 9 in. They have an absorption of about 8 per cent after immersion in cold water for 5 min, a transverse strength of 3,500 lb on a 7-in. span, and a compressive strength of from 8,000 to 9,000 lb per sq in. The mortar was composed of 1 part portland cement, $^1/_5$ part slaked lime, and $3^3/_5$ parts Mississippi River sand, all measured by volume. It was mixed in a batch mixer for at least 10 min before using. The average time of mixing for all mortar was about 30 min. Deformed rail steel was used throughout for the reinforcing bars except the $^1/_4$-in. round ties, which were smooth. The reinforcing steel was detailed and fabricated before shipment to the building site.

Methods of Construction

The reinforced brick columns were started from the concrete or brick footings and laid to a point approximately 6 in. below the finished exterior grade. The curtain walls were started from plank forms supported on dry brick piers set between the columns. It was necessary to use the plank forms because the original ground was about 2 ft below the finished floor grade. The curtain walls were bonded to the columns.

All the mortar joints have a minimum thickness of $^5/_8$ in. and are weather struck. The brick were laid

from a plank staging supported on wooden trestles. All brick were first thoroughly wet with a hose and then were laid by the Western or "stringing mortar" method, that is, mortar was spread along the top of the brickwork for 3 or 4 ft before the masons started to lay the next brick course. All the brickwork was laid with "shoved joints." Masons were not permitted to furrow the bed joints.

The reinforcing steel was set by the masons, who wired the vertical bars in the columns to the dowels in the footings and to a wood template about 10 ft above the ground. These bars were plumbed and then braced two ways with diagonal wooden braces. The column ties were put around the vertical bars before they were erected and the masons slid one tie down into a mortar joint when required. The horizontal wall bars were laid on the dry brick and were held in place by small pats of mortar until the next brick course was laid. All the brickwork was wet thoroughly twice a day for ten days after it was laid in order to facilitate the hydration of the mortar.

Details of Reinforcing Method for Brickwork

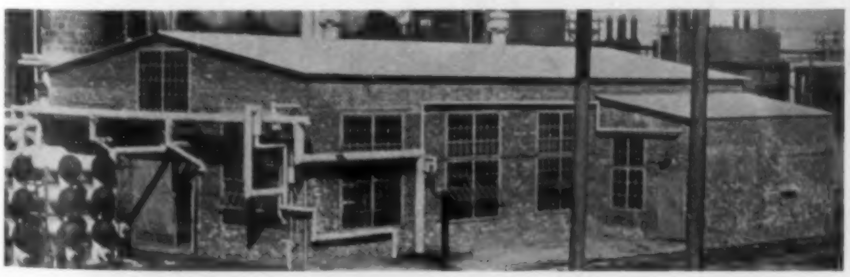
Columns Reinforced with Vertical Steel Attached to Dowels Projecting Above Their Concrete Footings; Walls Started on Plank Forms and Reinforced with Two Horizontal Bars Where 4 In. Thick, and with Six Bars Where 14 In. Thick

METHODS OF REINFORCING

The side columns of the main building are 23 by 27 in. up to the crane girder seat, above which they are 23 by 14 in. The reinforcing consists of six $^3/_4$-in. round bars, three in the outside and three in the inside of each column, located about $4^1/_2$ in. from each face, as shown in the illustrations. The three inside bars extend from the footing to 3 in. below the crane girder seat.

Three $^3/_4$-in. round dowels were set $4^1/_2$ in. from the inside face of the 14-in. section of each column so as to project 2 ft 6 in. above and below the crane girder seat. The bars in the column above this point were wired to these dowels. The eight horizontal mortar joints immediately below the crane girder seats were reinforced by placing rectangular pieces of welded wire mesh in the mortar as the brick were laid. The dimensions of the mesh were about 3 in. less than the dimensions of the column, and the wires were No. 8 gage, spaced 4 in. from center to center both ways. Throughout the remainder of the column $^1/_4$-in. round ties were used, spaced about $9^1/_2$ in. from center to center.

At the crane girder seat, a $^3/_4$-in. steel plate was cut to fit around the three dowel bars and set to grade in mortar.



Completed Building, with Reinforced Brick Walls

Standard Oil Company's Compressor House at Wood River Refinery, Illinois

The crane girder was electrically welded to this plate. A similar procedure was followed for the bearing plates of the roof trusses. About 6 in. of the outside ends of the roof trusses were bricked into these columns.

The columns at the sides of doors and along the outside of the switch room are 14 by 23 in. and were reinforced with four $^3/_4$-in. round vertical bars and $^1/_4$-in. round ties spaced about $9^1/_2$ in. from center to center.

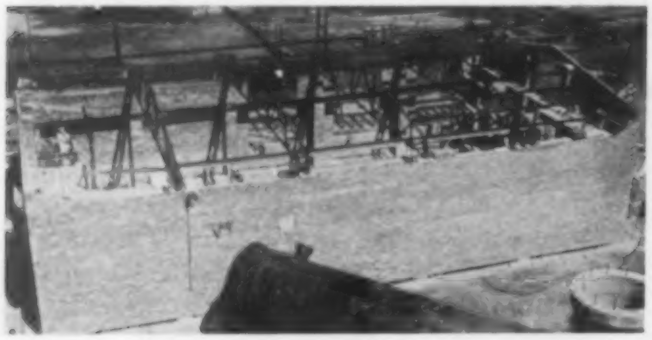
The walls in the two bays between the switch room and the main building contain three tiers of brick, and are approximately 14 in. thick. The horizontal mortar joint above the first course of brick is reinforced with six $^3/_8$-in. round bars, the outside bars being about $^1/_2$ in. from each face of the wall. Above this point, two $^3/_8$-in. round bars were used in each second horizontal joint, at a vertical spacing of about $6^3/_8$ in. from center to center. These bars were set in the mortar joint about $^1/_2$ in. from each face of the wall, which was made quite heavy because it was anticipated that the switching equipment would cause considerable vibration.

All other walls contain only one tier of brick, and are nominally 4 in. thick. The horizontal mortar joint above the first course of brick is reinforced with two $^3/_8$-in. round bars, placed about $^1/_2$ in. from each face of the wall. Above this the reinforcing is in every third horizontal mortar joint, at a vertical spacing of about $9^5/_8$ in. from center to center. To provide for positive moment due to wind loads, one $^3/_8$-in. round bar was placed about $^1/_2$ in. from the inside face of the wall and extended between the brick columns. To provide for negative moment at the columns, bars with a length of one-half the span were set so that their centers would come at the center of the columns. This gave negative reinforcement for one-quarter of the span. These bars were located about $^1/_2$ in. from the outside face of the wall. At the corners of the building, one $^3/_8$-in. round bar was set $^1/_2$ in. from the outside face of the wall and extended 4 ft along each wall. In the first joint above the head of windows and doors, two $^3/_8$-in. round bars were set, in order to provide reinforced brick masonry lintels.

The frames for the doors and windows were made from 3 by 3 by $^1/_4$-in. steel angles, and the side angles were punched with slotted holes 19 in. from center to center, so as to coincide with every sixth mortar joint in the brickwork. Carriage bolts, $^3/_8$ by 6 in., were put through these holes and bonded into the mortar joint as the brickwork progressed. Stock steel factory sashes and mullions were welded to the window frames.

The cost of this building, exclusive of the foundations for the compressors, the 12-ton traveling crane, the heating equipment, and the electric wiring, was 9.2 cents per cu ft, or $1.875 per sq ft. The compressor house was

only a small part of the construction that was being carried on in that part of the refinery. It was necessary to put up this building in parts so that temporary railroad



COMPRESSOR BUILDING NEARING COMPLETION, 1932

tracks and other construction equipment could be used. A comparison of the costs of various types of curtain walls used at this refinery is given in Table I.

TABLE I. COST OF VARIOUS TYPES OF CURTAIN WALLS

Type	Cost per Sq Ft
4-in. reinforced brick, using No. 2 pavers (columns not included)	23.0 cents
Corrugated asbestos and steel girts (columns not included)	27.0 cents
8-in. plain common brick	47.5 cents
6-in. reinforced concrete, using metal panel forms	56.3 cents

On the morning of July 26, 1932, rain to the amount of 1.49 in. fell between 4:00 and 7:00 a.m. The west wall of this building was wet on the inside from a point about 12 ft above the floor down to the floor. No other parts of the building seemed to be affected in this way. Investigation revealed that water was running off the roof, which has no gutter at the eave, and falling on to some large pipes, the tops of which are about 13 ft above the floor. The water splashed from the pipes on to the 4-in. brick walls and apparently seeped through the mortar joints. Several methods of remedying this situation were suggested, among them: (1) to provide a gutter along the eaves; (2) to waterproof the exterior of the brickwork with a colorless waterproofing compound; and (3) on future work, to use a tooled concave joint in the brickwork.

The leakage has now practically stopped, with no changes or additions to the original construction. Apparently, the water seeping through the joints has tended to "heal" the cracks through which the water was leaking. On other buildings, the third of which was completed in February 1933, concave tooled joints were used. There has been no evidence of leakage on any of these buildings. However, none of them has been subjected to as severe conditions as the compressor house.

It appears to me, after my experience with this building, that the use of reinforced brick masonry can be expected to become quite general. This is especially true of industrial work because it is economical to construct; it presents a more pleasing appearance than corrugated iron or corrugated asbestos; the heat loss through a 4-in. brick wall is about 60 per cent of that through corrugated asbestos; and its maintenance will probably be small. It also appears that reinforced brick masonry may be expected to supplant reinforced concrete in many places where there are large vertical surfaces, because it obviates the necessity of providing forms for such surfaces.

Acknowledgment is made to the management of Standard Oil Company (Indiana) for permission to publish this article.

OTHER EXAMPLES OF REINFORCED BRICK CONSTRUCTION

GRADE SEPARATION IN PERLA, ARK.

The abutments and wing walls are of reinforced brick; the latter have a maximum thickness of 2 ft and a maximum height of 16 ft.

REINFORCED BRICK ARCH OF 42-FT SPAN, PHILADELPHIA

Designed to carry a live load of 80 lb per sq ft, this arch was built without falsework, on movable centers, and each half of the arch ring was cantilevered from its abutment until final closure was effected at the center. The rise of the arch is 6 ft. The arch barrel is 8 in. thick and 4 ft wide, and has 8-in. spandrel walls, which support deck slabs 12 ft wide, of precast reinforced brick. Thus the floor slab cantilevers 4 ft beyond the spandrel walls on each side. The structure is situated on the grounds of the Melrose Country Club, near Philadelphia.

Photographs courtesy of the Common Brick Association

Distribution of Pressure Under a Footing

Preliminary Tests of Reactions Under a Square Concrete Block Reveal Non-Uniform Distribution of Load

By Frederick J. Converse

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Instructor, California Institute of Technology, Pasadena

Interest in the results of the test described here by Mr. Converse lies not only in the development of a plunger type of pressure cell but also in the results obtained from the readings, which show the distribution of the pressure under a test footing. An applied load on the footing equivalent to a uniformly distributed load of 1.5 tons per sq ft resulted in a maximum concentration of 1.92 tons per sq ft under the center of the footing and a minimum of 0.65 ton per sq ft on the diagonal at a point halfway between the center and the corner. With an applied load of 2.65 tons per sq ft, the maximum concentric load was 4.2 tons per sq ft at the center, which decreased uniformly to 0.5 tons per sq ft at the corner. Another interesting result of these tests was that friction between the footing and the soil into which it was poured accounted for as much as 45 per cent of the total applied load.

At the California Institute of Technology, a series of tests were recently conducted, designed primarily to assist in the development of pressure cells of the plunger type, for use in determining the distribution of pressure under building footings. However, such accurate results were obtained regarding the distribution of soil pressure under the test block as to make the tests worth while also in the field of soil pressure research. The work was done at the California Institute of Technology, under my direction.

The test block was prepared by digging a hole in the ground 4 ft square and 4 ft deep. At the bottom of this hole, and concentric with it, another hole 3 ft square and 10 in. deep was excavated. Eleven pressure cells were set in place in the pit with the ends of the cylinders resting on the soil, and concrete was poured around them to a depth of 10 in. The cells were arranged as shown in the photograph, Fig. 1, on which the various dimensions are marked.

PRESSURE CELLS OF RUGGED DESIGN

The design of the pressure cells used in the experiments was the result of an attempt to obtain a pressure measuring device that could be left in place for long periods of time without damage, and which would indicate pressure at any time without requiring much movement of that part of the cell in contact with the soil.

A vertical section of the cell used in the experiments is shown in Fig. 2. The barrel of the cell consisted of a piece of 3-in. standard iron pipe 14 in. long, threaded at one end and faced square at the other. Inside this, a cylindrical piston 3 in. in diameter and 3 in. long was fitted loosely, one surface being flush with the faced end of the pipe. A piston rod $^3/_4$ in. in diameter, threaded for its full length, was screwed into the opposite face of the piston. This rod was of sufficient length to extend about 5 in. beyond the open end of the pipe. Two hexagonal nuts were screwed on the piston rod and were locked in place when the outer face was within about $^1/_4$ in. of being flush with the threaded end of the pipe. A standard cast-iron pipe cap, having a hole drilled in the center large enough to allow the piston rod to pass through it, was screwed on the threaded end of the pipe until the nut on the piston rod pressed against the inside of the pipe cap. A nut screwed on the piston rod outside the pipe cap, locked the piston, pipe cap, and pipe together, so that they formed a rigid unit. The faced end of the pipe and the working surface of the piston were then sealed against the entrance of dirt and water by a strip of brass 0.003 in. thick, placed over the end and soldered to the pipe.

When thus assembled, the unit was ready to be placed in the concrete footing. The pipe was placed vertically on the soil, with the sealed end down, and concrete was poured around it to within a few inches of the pipe cap. As load was placed on the footing, the pressure on the piston was transferred through the piston rod and the nuts to the pipe cap, thence to the pipe and, by bond, to the concrete. The magnitude of this pressure was obtained by loosening the outer nut, and then loading the top of the piston rod until the pressure between the inner nut and the pipe cap was relieved. All the load on the piston was thus carried by the thrust on the top of the rod. A slight increase in this thrust was sufficient to push the piston into the soil, and to cause movement between the piston rod and the pipe cap. The amount of this movement should be small. In the tests in question a movement of 0.0001 in. was used, as indicated by a dial gage attached to the pipe cap and making contact with the piston rod.

The thrust on the top of the piston rod was provided

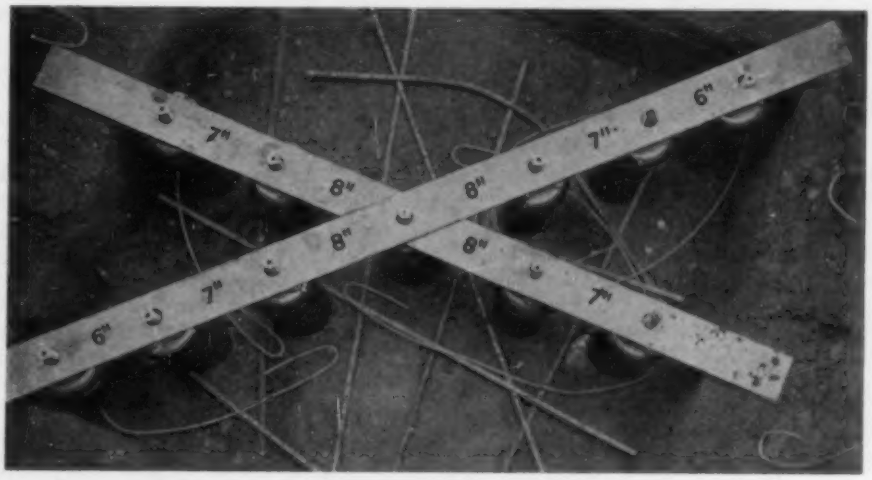

Fig. 1. Pressure Cells in Place in the Test Pit
Before the Footing Was Poured

by a lever having a ratio of 10 to 1. One end of the lever was fastened to the pipe cap by a chain and screw, and the opposite end carried a pan in which shot could be placed.

While this apparatus gave fairly consistent results, it was found desirable to modify some of the details of the design. The main feature of the new design is a loading system in which there will be no side thrust on the piston, and in which all possibilities of friction are eliminated.

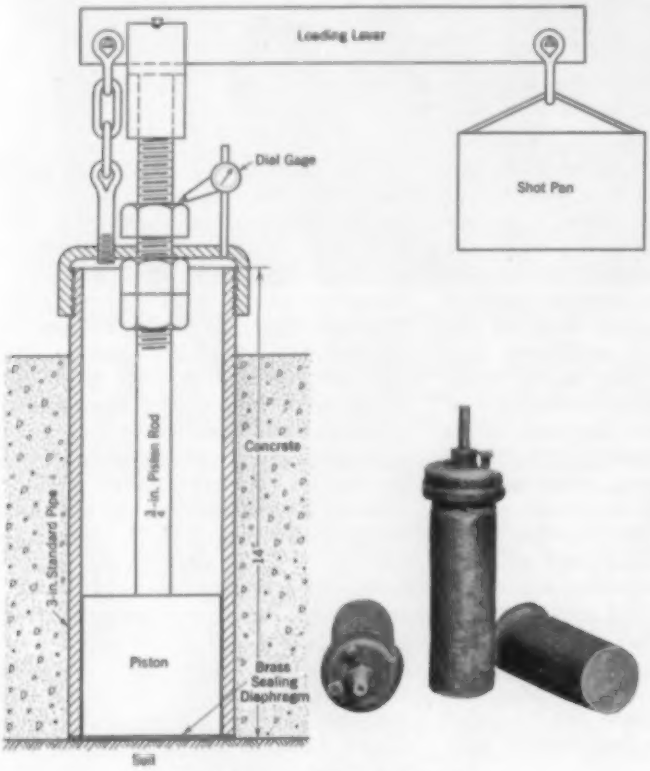

FIG. 2. DETAILS OF PRESSURE CELL
Pressure Measuring Lever in Place

The soil on which the concrete block rested was a clay-loam, according to the classification of the U.S. Bureau of Public Roads, and was made up of 29 per cent clay, 30 per cent silt, and 41 per cent sand. The weight of the dry soil was 90 lb per cu ft. The percentage of moisture when the tests began was 14 per cent by weight of the dry material, and at the end of the tests was 9 per cent.

TEST FOOTING LOADED ACCORDING TO RECOMMENDATION OF SOCIETY'S COMMITTEE

Load was applied to the concrete block by means of a lever and weights. A 15-in. I-beam was used as a lever, and the column transferring the lever pressure to the block was an 8-in. standard iron pipe, having special ball and socket ends. The base of the column rested on steel plates supported by the concrete. An I-beam placed at right angles to the lever, and fastened to two large sand boxes, supplied the lever reaction. For the load, standard 50-lb weights were used, suspended in a cradle from the free end of the lever. This apparatus is similar to that recommended by the Society's Committee on Bearing Value of Soils, as described in PROCEEDINGS, Vol. 46, Papers and Discussions, page 905 (August 1920).

Twenty-five days after the concrete was poured, an initial load of approximately 1.5 tons per sq ft was applied, with the result that a settlement of about 0.1 in. occurred. Readings of pressure on the cells were taken the day the load was applied and at intervals for the next 10 days. Four sets of pressure readings were observed at this load. From these results, the iso-pressure lines in Fig. 3(*a*) were plotted. A curve showing the pressure along a line through the center perpendicular to the side is shown in Fig. 3(*b*), and a curve of pressures along a diagonal of the footing is given in Fig. 3(*c*). From these curves it is evident that concentrations of pressure occurred at the corners and at the center under this load, the pressure at the center being roughly twice that at the corners, and the minimum pressure being roughly two-thirds the pressure at the corner.

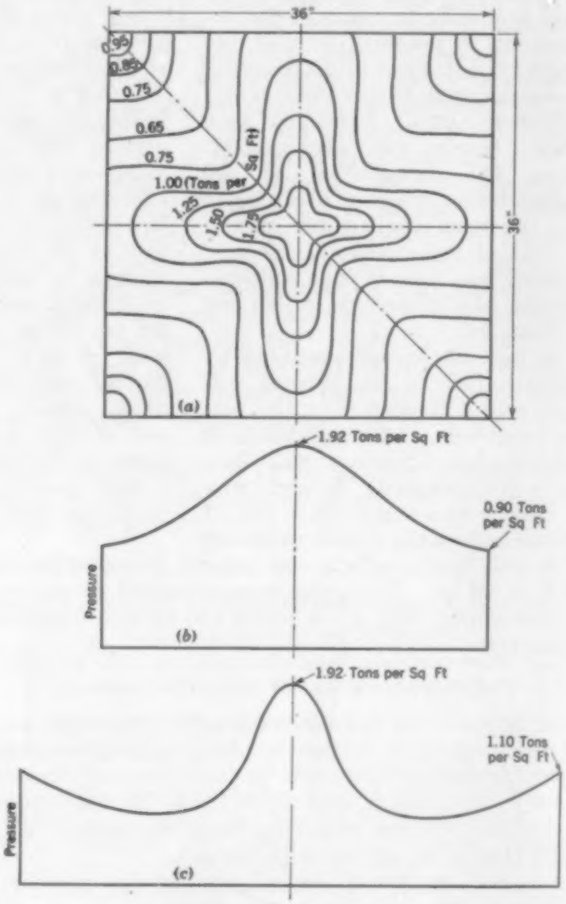

FIG. 3. SOIL PRESSURE DISTRIBUTION FOR AN APPLIED LOAD OF 1.5 TONS PER SQ FT

(*a*) Iso-Pressure Lines Under a Square Concrete Block
(*b*) Pressure Distribution Along the Center Line
(*c*) Pressure Distribution Along the Diagonal

Forty-eight days after the concrete was poured the load was increased to 2.65 tons per sq ft, and readings of pressure on the cells were taken 8 days and 30 days after the increased load was applied. The results of the readings taken 8 days after the increase in load are shown in Fig. 4(*a*) and (*b*), by the curves marked (2). In this case the concentrations of pressure at the corners no longer existed, and the tendency was toward a more uniform distribution of pressure.

Before the 30-day readings were taken, the soil at the sides was loosened to a depth level with the bottom of the block, thus eliminating the effects of side friction. Curves marked (1) in Fig. 4(*a*) and (*b*) show the effect of removing this friction. The distribution is, in general, similar to that indicated by the curves marked (2), taken

before the side friction was removed, but the pressures are greater. In Fig. 5 are shown the iso-pressure lines for this case. The general tendency toward more uniform distribution of load is evident.

Load Carried by Friction on the Sides

In comparing the total load on the concrete block, as measured by the pressure cells, with that calculated from the loading beam, it was apparent that all the load was not being transferred to the soil through the base of the block. For the load of 1.50 tons per sq ft, the pressure cells indicated only about 65 per cent as much load as was actually applied. The rest was undoubtedly carried by friction on the sides of the block. Similarly,

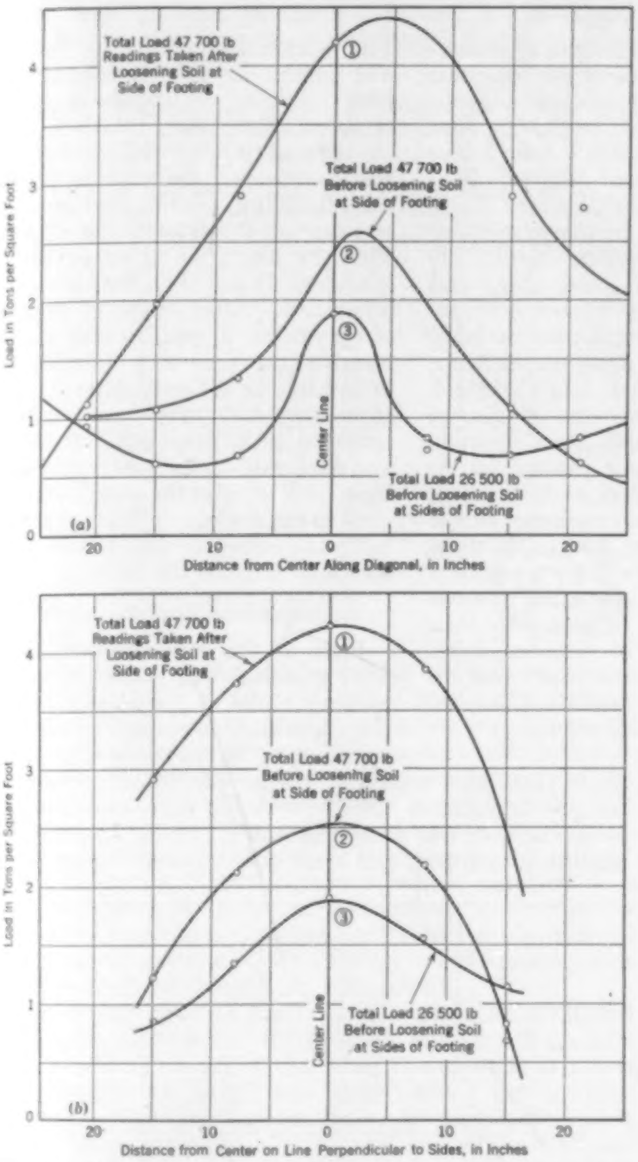

Fig. 4. Curves Showing Load-Carrying Effect of Friction on the Sides of the Footing

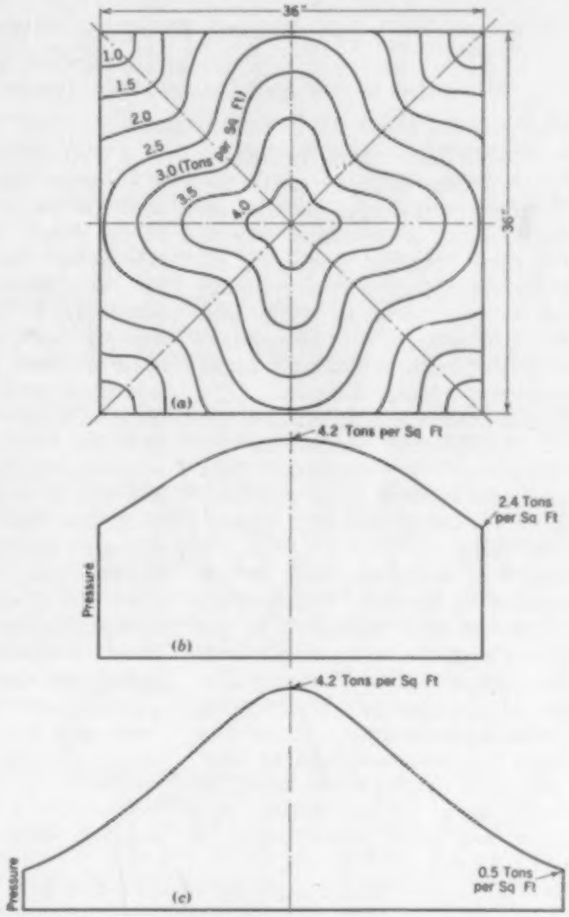

Fig. 5. Soil Pressure Distribution for an Applied Load of 2.65 Tons per Sq Ft

(*a*) Iso-Pressure Lines Under a Square Concrete Block
(*b*) Pressure Distribution Along the Center Line
(*c*) Pressure Distribution Along the Diagonal

the first set of readings with the load of 2.65 tons per sq ft showed a total load equal to only about 55 per cent of that actually applied. Readings taken after the soil had been loosened around the sides of the block indicated a total soil pressure equal to about 95 per cent of the load applied.

Grateful acknowledgment is made of the cooperation of R. V. Labarre, M. Am. Soc. C.E., who furnished the equipment used in these tests.

Tablet on the St. Johns River Bridge at Jacksonville, Fla.

"Formerly the richest countries were those in which Nature was most bountiful; now the richest countries are those in which man is most active. For in our age of the world, if Nature is parsimonious, we know how to compensate for her deficiencies; our engineers can correct the error and remedy the evil. For the powers of Nature, notwithstanding their apparent magnitude, are limited and stationary; but the powers of man, so far as experience and analogy can guide us, are unlimited."

—*From the "History of Civilization," by Henry Thomas Buckle*

Effect of Different Media on Trickling Filter Unloading

By WILLEM RUDOLFS, M. AM. SOC. C.E. *and* NOEL S. CHAMBERLIN

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A PRODIGIOUS amount of investigation and research on materials used in filters—their efficiency, durability, and general usefulness for stabilizing sewage—has been done by the Committee on Filtering Materials of the Society's Sanitary Engineering Division. In the past it has been difficult to compare results obtained at one plant using slag, with those at another using crushed stone, for example, because of such other variables as strength of sewage, climatic conditions, and rate of application. At Plainfield, N.J., the State Department of Sewage Research has eliminated these variables by applying the same sewage at the same rate of application on three filters, identical except that in one case crushed stone, in another, slag, and in the third, gravel, was used as the filtering medium. After 18 months of operation, samples from each 1-ft layer of each filter were taken, the amount of retained solids measured, and a complete analysis made. Dr. Rudolfs and Mr. Chamberlin here give the results obtained.

ENGINEERS and operators have long debated as to the effect of different media on the unloading of trickling filters, basing their opinions in general on results obtained at different plants where factors such as type and strength of sewage, rate of application, and climatic conditions make comparisons rather difficult. To eliminate these variable factors, the effect of different media on the stabilization of settling tank effluent was studied by us at Plainfield, N.J., using identical experimental units.

The filter materials used in our experiments—crushed stone, slag, and gravel—were specified in accordance with the recommendations of the Committee on Filtering Materials of the Sanitary Engineering Division of the Society. According to these recommendations, at least 95 per cent of the stone must be retained on a $1^1/_2$-in. screen, and 100 per cent must pass through a $2^1/_2$-in. screen.

Each filter unit had a capacity of approximately 132 cu ft and was 6 ft deep. During these experiments each unit received 1,190 gal of sewage daily through 180 siphon spills—a flow corresponding to 1.96 mgd per acre. The nozzle supplying the sewage was placed in the middle of the filter. During an operation period of about 18 months, stabilization of the effluent from the different media was about equal, except that during the sloughing periods the amount of suspended solids in the effluent from the slag filter appeared to be considerably less than that from the other two. After these 18 months of operation, it was decided to determine the amounts of solids retained by the different filters. The stone, slag, and gravel were carefully removed in layers, each 1 ft thick, and duplicate samples were collected from each layer in the zone receiving most of the sewage. This was about halfway between the nozzle and the outer edge of the tank.

COMPLETE ANALYSES MADE

Each of the duplicate samples of filter material was placed in a pail holding about 1.2 gal and transported to the laboratory. Analyses were made to measure the total solids, ash, and nitrogen present and to determine the oxygen consumed. For the latter determination, the entire sample was placed in water, rotated in a shaking machine, scrubbed, and then washed once. Four anal-

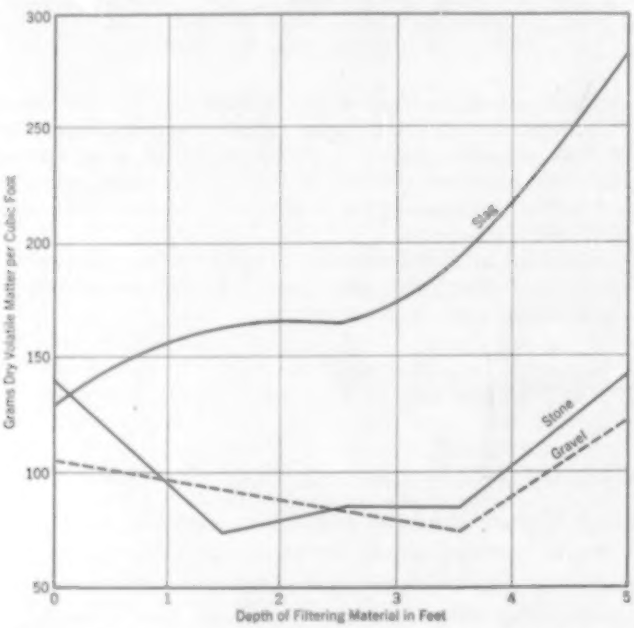

FIG. 1. DISTRIBUTION OF DRY VOLATILE MATTER PER CUBIC FOOT OF FILTER MATERIAL

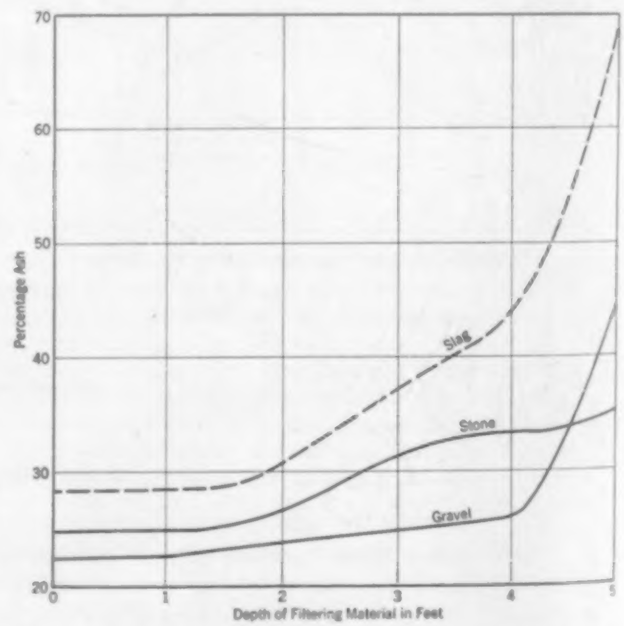

FIG. 2. ASH CONTENT OF SLUDGE AT DIFFERENT DEPTHS IN THE EXPERIMENTAL FILTERS

yses were made on each sample, or 8 for each level, making a total of 40 determinations for each filter media. To determine the amount of solids and ash, the washings were evaporated and combined for each level. The total nitrogen present was determined from the solids. The percentage of voids in each medium was also determined in the usual manner by the replacement method.

The results obtained, calculated on the basis of 1 cu ft of filter medium for each 1-ft layer, are shown in Table I. In the stone and gravel filters the total dry solids were greatest in the 0–1 ft and 4–5 ft layers, whereas in the slag filter the amount of dry solids increased downward.

TABLE I. MATERIALS PRESENT AT DIFFERENT LEVELS, PER CUBIC FOOT OF FILTER MEDIUM

Medium	Level Feet	Total Dry Solids Grams	Total Dry Volatile Matter (Fig. 1) Grams	Ash (Fig. 2) Per Cent	Oxygen Consumed (Fig. 3) Grams	Total Nitrogen Per Cent	Total Nitrogen Grams
Stone	0–1	159	119	25.0	47.3	5.6	8.96
	1–2	103	77	25.4	43.5	5.4	5.55
	2–3	117	83	29.0	44.9	5.4	6.32
	3–4	124	83	33.2	38.2	4.6	5.72
	4–5	184	121	34.0	46.1	4.5	8.28
Slag	0–1	202	145	28.3	65.1	5.6	11.12
	1–2	226	162	28.5	66.5	5.3	12.03
	2–3	251	165	34.2	72.6	4.9	12.30
	3–4	318	191	40.0	73.7	4.2	13.44
	4–5	581	252	56.7	77.5	2.2	12.78
Gravel	0–1	131	101	22.6	48.4	6.0	7.86
	1–2	117	89	24.0	43.7	5.7	5.66
	2–3	106	81	23.8	44.8	6.4	6.78
	3–4	96	72	24.7	36.7	6.1	5.98
	4–5	159	104	34.8	51.8	4.7	7.52
Averages:							
Stone	0–5	137	96	29.8	44.0	5.1	6.97
Slag	0–5	315	183	42.0	71.1	3.92	12.33
Gravel	0–5	122	89	27.0	45.1	5.55	6.76

There was a marked difference in the ash content of the sludge held in the different filters. The average oxygen consumed of the sludge in the slag was 38 per cent greater than in the stone, while the total nitrogen content was 43 per cent higher. The distribution of the dry volatile matter in the different filters is graphically shown in Fig. 1. The volatile matter in the slag increased rapidly from the $2^1/_2$-ft level downward, indicating a filling up of the interstices which occurred to a far less extent in the stone and gravel filters. In the stone and slag filters a narrow zone near the top retained the coarser materials, where they were disintegrated and partially destroyed.

The upper part of a filter acts mainly as a strainer and is the place where the greatest numbers of bacteria are found. Just below this zone the greatest numbers of protozoa are found, and these reduce the numbers of bacteria by feeding on them, as was indicated in this experiment especially by the enormous reduction in B. coli. Below the 2-ft level the oxidizing bacteria begin to play a more important rôle, the ammonifying organisms are reduced, and those forming nitrites and nitrates increase. It may be assumed that the increase in solids in a filter bed would eventually clog it so that its efficiency would be impaired.

RESULTS DIFFERED FOR DIFFERENT MEDIA

Calculations of the total solids, volatile matter, and ash of the solids found in the different filters are given in Table II. The amount of dry solids in the slag filter was 228 per cent higher than that in the stone filter in spite of the fact that the percentage of voids was only slightly greater. The percentage of volatile matter, however, was 188 per cent greater in the slag than in the stone, indicating either that more breakage had occurred in the slag or that the organic matter in the slag had been more completely oxidized.

TABLE II. SOLIDS PRESENT IN EXPERIMENTAL FILTER BEDS

Medium	Total Dry Solids In Pounds	Dry Volatile Matter In Pounds	Ash In Per Cent	Total Nitrogen In Grams	Oxygen Consumed In Pounds	Voids In Per Cent
Crushed stone	40.2	28.2	29.6	92.8	12.7	49.8
Slag	92.0	53.2	42.0	185.0	21.3	52.6
Gravel	35.6	26.0	26.8	93.8	13.2	44.5

Plotting the ash content of the solids present throughout the filters (Fig. 2) showed a persistently higher percentage in the slag than in the stone, increasing very rapidly towards the bottom. A small part of the higher ash percentage was undoubtedly due to a greater extent of oxidation of the organic matter, but calculations showed that more of it was due to the presence of inert matter. Visual observations indicated also a rather large amount of fine, sharp, gritty material, grayish-black in color, which resembled finely crushed slag.

Further evidence of the presence of larger quantities of inert material in the slag filter was brought out by plotting the values for oxygen consumed by the material in the different layers of the filters (Fig. 3). The difference between the slag and the stone was comparatively small, indicating that the material was somewhat more oxidized in the slag filter, but not in proportion to the larger quantities of inert matter present. The actual total amounts of oxygen consumed in the experimental filters were as follows:

Stone filter	3,750 grams
Gravel filter	5,980 grams
Slag filter	9,650 grams

It is of interest to note that in the stone filter the zone of greatest putrescibility was in the 1 to 2-ft layer, whereas in the gravel and slag filters it was at the 2 to 3-ft level. The cause of the greater putrescibility in these particular layers is the breaking down of the coarser solids sprayed on the bed when strained out, together with the large numbers of protozoa and small animals present.

The percentage of total nitrogen present in the solids in the different layers remained fairly constant throughout the beds, except near the bottom, where it was considerably reduced. The different types of nitrogen sprayed on the beds, principally ammonia and organic nitrogen, underwent changes that are not reflected in these data. These changes have been discussed elsewhere, but it is of interest to consider here whether more of the nitrogenous matter was present in the liquid or the solid part of the applied sewage.

TABLE III. DISTRIBUTION OF NITROGEN IN LIQUID AND SOLID PARTS OF TANK EFFLUENT

Item	Amount
Total solids in liquid part	576 ppm
Volatile matter in liquid part	276 ppm
Total solids in solid part	4.17 per cent
Volatile matter in solid part	3.03 per cent
Total nitrogen in liquid	35 ppm
Ammonia-nitrogen in liquid	25 ppm
Nitrogen in volatile matter of liquid part	12.7 per cent
Nitrogen in solids	144 ppm
Nitrogen in volatile matter of solid part	0.47 per cent
Nitrogen in mixture	41 ppm

As an illustration, the distribution of the nitrogen in the liquid and solid part of the tank effluent sprayed on the filter is given in Table III. It is clear that by far the greater part of the nitrogen applied was present in the liquid sewage in the form of ammonia, and that the rest was in the form of organic nitrogen

in the finely divided and colloidal material. The volatile matter of the coarser solids contained only 3.03 per cent of nitrogen, whereas the finely divided volatile solids in the liquid contained 12.7 per cent.

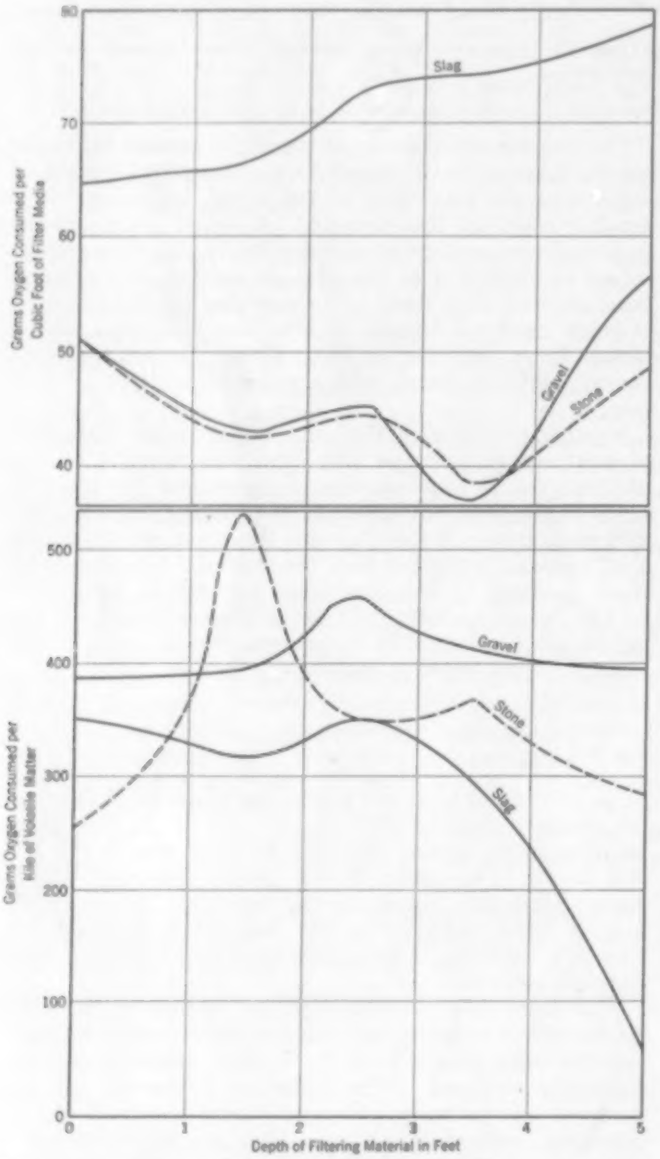

FIG. 3. TOP: PUTRESCIBILITY OF VOLATILE MATTER IN THE DIFFERENT FILTERS. BELOW: RELATIVE PUTRESCIBILITY OF EQUAL AMOUNTS OF MATERIAL TAKEN AT DIFFERENT DEPTHS IN THE THREE TEST FILTERS

This division is probably the cause of the fairly equal distribution of the nitrogen in the beds.

The results reported show the distribution of the solids and volatile matter in the filter beds in such a way that the more or less generally recognized functions of a trickling filter—to strain, to reduce, and to oxidize—are brought out sharply. The upper part of the beds acted as a straining and reducing device, while the lower part oxidized the material in solution, the finely divided suspended substances present in the liquid sprayed on the bed, and the products of decomposition or reduction washed down from the upper layers. It is therefore natural that, if the lower part of the bed accumulates large quantities of solids which are not regularly unloaded, the efficiency of the bed eventually will be reduced. The rate of the decrease in efficiency will depend on the length of time the bed has been in service and the load placed on it. This would indicate that the load on a sprinkling filter cannot be measured alone in pounds of biochemical oxygen demand applied to the bed, but that the quantity and possibly the size of the suspended matter must be taken into consideration.

The solids to be unloaded begin to accumulate near the bottom of the bed, gradually filling up more and more of the voids. It is in this accumulation that the number of worms is greatest. These work over the mass and are undoubtedly one of the factors that cause the sloughing off. As has been demonstrated elsewhere by the senior author, sloughing starts near the top, in the layer that strains out the coarser material, so that these solids are added to those already present near the bottom, which have been gradually washed down. The actual sloughing would appear to be caused by a combination of factors, such as worms, amount of solids, pressure of water after a spray on the accumulated solids near the bottom, and relative roughness of material. The problem of inducing sloughing artificially is of considerable interest in this connection.

The fine inert material present in the bottom of the slag filter was probably caused by the wearing down of the sharper edges of the slag rather than by disintegration of the larger pieces. During the first three months of operation there was a continuous discharge from the slag filter of very fine black material, which gave the effluent a decided color. This material was present to a much less extent in the effluent from the crushed stone filter, and was practically absent in that from the gravel filter. The gritty, inert material retained near the bottom of the slag filter was apparently too heavy to be washed out.

PRELIMINARY SETTLEMENT IMPORTANT

It is of decided practical interest that the nitrogenous material sprayed on the bed was present mostly in a liquid form, showing once more that a settling tank is a decidedly advantageous segregating device. The greater the amount of the solids retained in a settling tank, the more nearly a trickling filter becomes an oxidizing device.

It may be of interest to translate the differences in solids found in the experimental slag and stone filters into terms of a trickling filter in practical operation. Calculating the results on the basis of the Plainfield, N.J., trickling filters, receiving about 3.3 mgd of sewage, the dry solids in the stone filter would amount to 69.3 tons, whereas those in the slag would be 160.6 tons. On the basis of 5 per cent sludge, this would be respectively 22,400 and 52,000 cu ft, occupying about 20 per cent of the total voids in the stone filter and about 45 per cent of those in the slag.

"Therefore when we build, let us think that we build forever. Let it not be for present delight, nor for present use alone. Let it be such work as our descendants will thank us for, and let us think, as we lay stone on stone, that a time is to come when those stones will be held sacred because our hands have touched them, and that men will say as they look upon the labor, and wrought substance of them, 'See this our fathers did for us.'"—*Ruskin*

General View of the Bridge on Project 12, near Augusta, Ga.

Temperature Effects in Concrete Sections

Unequal Changes in Unsymmetrical Viaduct Slab and Girder Sections Create Serious Stresses

By Searcy B. Slack

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The study of stresses and deflections in concrete due to temperature changes is by no means a neglected subject. Yet in this paper Mr. Slack is able to throw new light on it in detailing the results of tests recently carried out on Georgia highway structures. The particular value of these findings lies in their application to construction that combines relatively heavy structural parts with light ones, such, for example, as the typical slab-and-girder viaduct. Fairly simple tests on two such structures indicated that serious stresses result from merely normal temperature variations. The fact that such stresses may approximate 5,000 lb per sq in. is probably not generally realized. Consideration of Mr. Slack's findings should have a beneficial effect on the future design of slab-and-girder construction.

A LARGE amount of study has been given to temperature stresses in concrete sections, but in the majority of the investigations and discussions on this subject the assumption has been made that the temperature distribution throughout the concrete section was approximately uniform. That this assumption is in error has been clearly shown by the well proved curling action of concrete pavements and by the differences in temperature reported between surface and interior sections of concrete arches and dams.

Measurements previously made indicate that the temperature of comparatively thin concrete slabs follows very closely the temperature of the air. In heavier sections, however, it lags behind the air temperature. A quick change in the temperature of the air or slab therefore causes a considerable difference in temperature between the slab and the heavier girder section. Even ordinary changes in the temperature of an exposed structure produce appreciable deflection and stress. The data here presented and discussed show the effects of differences in temperature between a concrete slab and a girder section, as generally used in construction.

Field Measurements Taken in Georgia

The first field measurements were made on Project 220, which is a reinforced concrete trestle near Gainesville, Ga. As shown in an illustration, the structure consists of a series of deck girder spans. The decks, which are 38 ft long, have four girders and a slab of usual design and are supported on concrete bents. The measurements here reported were made on the two inside girders of spans Nos. 13 and 14. These girders were selected because they would be least affected by the curb section and reinforced concrete rail. The decks were covered with a plain concrete pavement 4 in. thick.

After the measurements on Project 220 were completed, it was decided to make a set of similar measurements on Project 12, which is another reinforced concrete trestle, located near Augusta, Ga. This project was selected because it also consists of a series of concrete deck girders 40 ft long, but without a concrete pavement over the deck slab. As on the other structure, measurements were made on the two inside girders, using in this case spans Nos. 3 and 4. The general features of the trestle on Project 12 are clearly indicated in the photographs and in Fig. 1. The concrete was proportioned by weight, with an approximate mix of $1{:}2{:}3^1/_2$, using sand and granite aggregates.

Temperature measurements were made with ordinary laboratory thermometers placed in small holes drilled into the concrete slab and girders. The holes were closed with cork stoppers, one of which was fitted around the thermometer so as to close the hole while the reading was being taken. The bottoms of the holes were filled with cup grease. The location of the holes and the general dimensions of the concrete slabs and

beams on which the measurements were made, are shown in Fig. 2.

Deflections on Project 220 were measured between a pin grouted into a hole near the bottom of the girder and a pin driven into the side of an 8-in. pine pole, so arranged that the weight of the operator would not affect the readings. The poles were securely bedded in the ground on concrete footings at the lower ends and were well braced. These bents were located near each end of the deck girders and also at their centers.

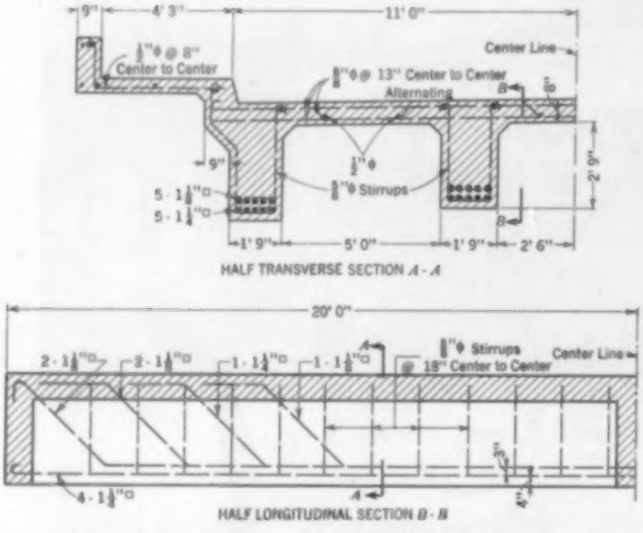


FIG. 1. DETAILS OF VIADUCT FLOOR, PROJECT 12

Deflection of the spans at the center was secured by measuring the distances between the pins with a micrometer screw, averaging the readings at each end of the girder, and then subtracting from the reading at the center of the girder. It was assumed that temperature changes would affect all the pine-pole bents alike.

On Project 12, two sets of measurements have been made, the first in April 1932 and the second in October 1932. On the first set, the temperatures were measured in the same way as on Project 220. However the deflection measurements were secured by fastening three 4-in. timbers to the sides of the girders. One timber was fastened at the center of the girder, and one was placed 6 in. from the point of support at each end of it. These timbers extended to within about 18 in. of the ground, and below each a heavy wooden stake was driven into the ground, as shown in a photograph. The stake was scabbarded in guides so placed that the timber supported from the beam could move up and down freely, but its lateral motion was restricted. One steel pin was driven into the stake and another into the timber fastened to the girder. The distance between these pins, measured with a micrometer screw, gave the relative elevations of the center of the girder and of its ends. As in Project 220, it was assumed that changes in air temperature and moisture would affect all the timbers alike.

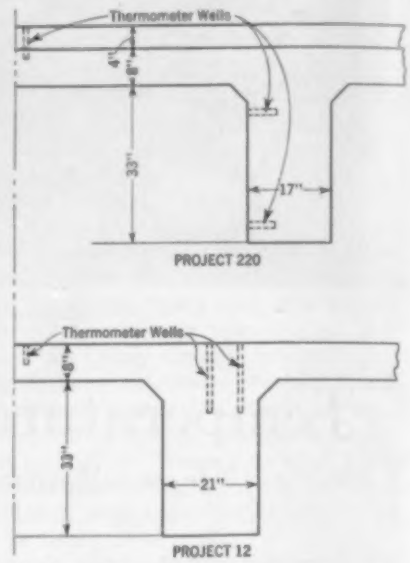
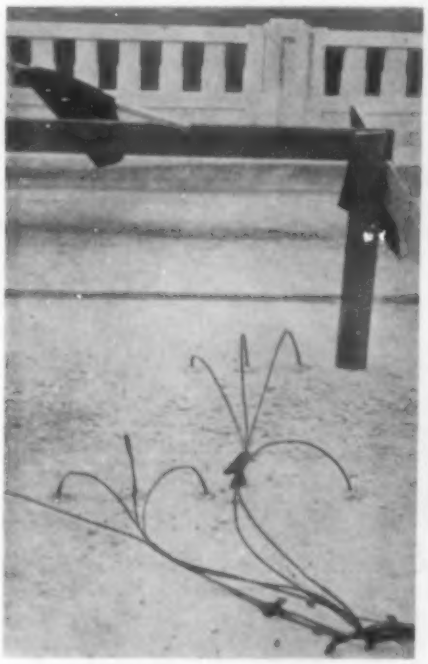


FIG. 2. CROSS SECTION OF PART OF VIADUCT STRUCTURES

Points Where Temperature Measurements Were Taken Are Indicated

In the second set of measurements on Project 12, the



Temperature Coils in Top of Slab and Girder

Timbers Clamped to Sides of Beams for Deflection Measurements, First Series

Wire Fastened to Girder Above for Deflection Measurements, Second Series

ARRANGEMENT OF TEST APPARATUS FOR MEASURING TEMPERATURE AND DEFLECTION, PROJECT 12

temperatures were measured by resistance coils placed in holes drilled into the slabs and girder. The deflections were measured by fastening wires to the girders. One wire was fastened to the center of the girder and one was placed 6 in. from the support at each end of the girder. These wires were of Invar steel having a very low coefficient of expansion. To the lower end of the wires were fastened heavy blocks of metal, as shown in a photograph. The lower face of these blocks was machine finished. Under each of the wires a heavy stake was driven into the ground, to the top of which was fastened a measuring dial. The plunger of the measuring dial, reading to 0.001 in., was adjusted against the bottom of the block of metal supported by the wire. The readings of these dials gave the relative elevations of the center of the girder and of its ends. Unfortunately there was very little change in temperature during the three days this equipment was maintained, so the temperature readings in the second group were not as significant as those in the first. These readings checked very well, however, with the readings in the first group. The strain gages used in connection with the second set of readings failed to work.

Readings taken on the two girders and on two spans of each project checked very well. Results are shown in Figs. 3 and 4. The data are presented as differences in temperature between the slab and the center of the girder, and the corresponding deflections at the center of the span. By this means consideration of the data is simplified.

Each point on the curves shown in Fig. 3 represents an average of four readings on two girders in two different spans. The curves for Spans 3 and 4 are given separately in Fig. 4 to show how closely the measurements checked. The points on these curves are the average of two readings for the two inside girders.

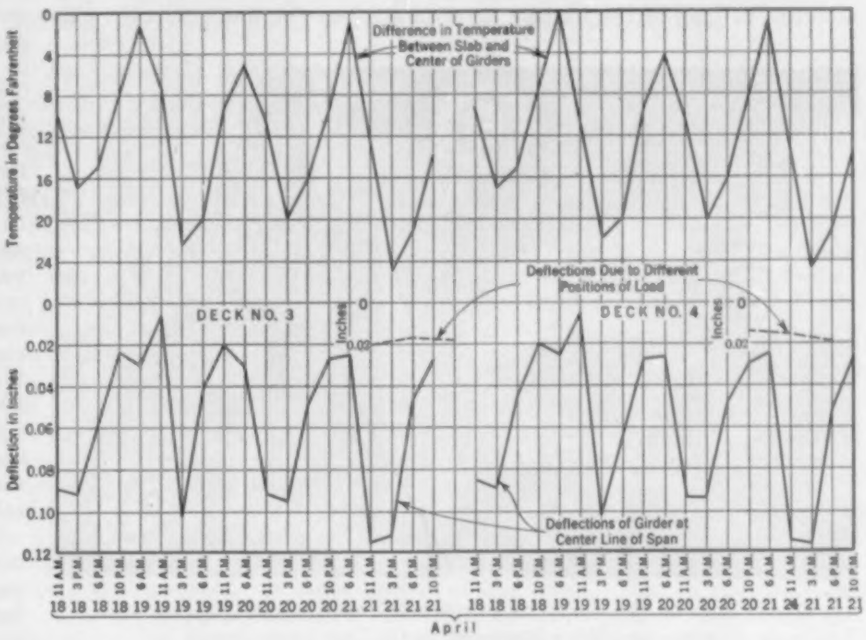


Fig. 4. Relation Between Temperatures and Deflections for Two Spans Separately

Points Represent the Average of Two Readings on Project 12

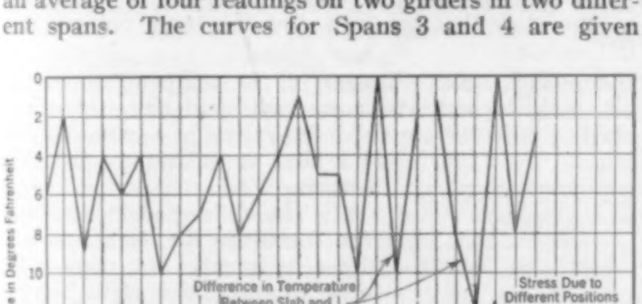


Fig. 3. Relation of Temperatures, Deflections, and Stresses

Points Represent the Average of Four Readings on Project 220

It will be noted that the measured deflections follow closely the difference in temperature between the heavy section of the girder and the slab; also that the measured deflection on Project 220, having a heavy slab, was roughly about half that on Project 12, where the slab was lighter. The general dimensions of the deck girders were similar, but the girder on Project 12 was 4 in. wider than that on Project 220. The data indicate that the average rise or fall due to one degree of difference in temperature between the center of the girder and the slab was 0.0045 in. on Project 220 and 0.00403 in. for the first set of readings on Project 12, and 0.0039 in. for the second set.

The effect of the extra pavement on Project 220 was to reduce the difference in temperature between the slab and the center of the girder section. The heat radiates very rapidly from the comparatively thin concrete slab, but not so rapidly from the heavy concrete girder. This causes a difference in temperature between the center of the concrete girder and the concrete slab.

Temperature measurements taken near the bottom of the girders on Project 220 were found to correspond very closely to those of the slab and the air. On Project 12, an additional hole was placed 4 in. inside the edge of the girders. The result indicated that radiation is not so rapid from the sides of the girder as from the slab, where both sides of the thin section are exposed.

In Fig. 5, a section of a concrete deck girder is shown, with its probable isothermic lines. Area B, area C, and a thin section along each side of the girder, are at approximately the same temperature, while area A may be at either a higher or lower temperature. Area B is much greater than area C; therefore the moment couple would result from the action of area A against area B, and the neutral axis would be shifted slightly on account of area C.

The deflections found were so large that naturally there was doubt as to whether or not corresponding stresses really existed in the steel. On Project 220 two electrical strain gages loaned by the Society were fastened to bars in the bottom layer of steel on the four girders. Other strain gages fastened to short pieces of steel bars were placed nearby, for use in checking the possible effect of temperature on the gages used for the actual measurement. A small box was fastened around each of the gage installations. After the instrument was adjusted in place, the box was filled with dry sawdust as a protection against changes in the temperature of the air.

UNDER SIDE OF THE DECK ON PROJECT 220, IN HALL COUNTY, GEORGIA
Deflections Were Measured on the Two Inside Beams

The strain gages were balanced to show zero strain at 5 p.m. on March 24, 1932, and corresponding temperature and deflection measurements were taken. Subsequent readings were recorded as shown in Fig. 3. It will be noted that the stress varied from an assumed zero at 5 p.m. to 2,400 lb per sq in. at 7 a.m. the next morning. Then there was a drop to a compression of 200 lb per sq in. at 3 p.m. and another rise to 2,300 lb at 10 p.m. on the night of March 25. The check gages indicated that the strain gages were not influenced by changes in temperature, so no corrections were necessary in the readings.

As a check on the deflection and strain-gage measurements, a truck weighing 21,000 lb was placed on the span in various positions during the afternoon of March 25. The truck was spotted so as to produce the maximum moment in each of the girders. It was found, however, that the 8-in. slab distributed the load throughout the width of the span, and there was little difference in the readings for the different girders. The resulting live-load stresses, which varied from 600 to 1,000 lb per sq in., and the corresponding deflections are shown in Fig. 3. The measurements indicate that the stress due to variation in temperature was more than twice as much as that caused by a 21,000-lb live load.

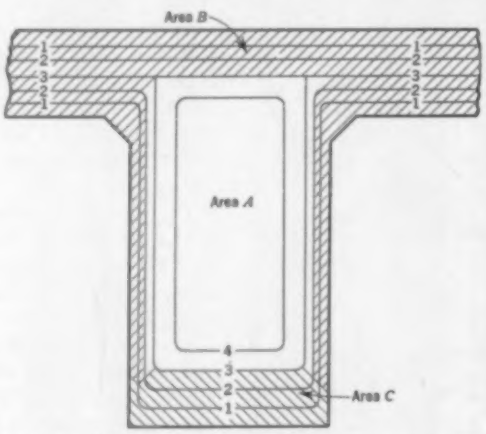


FIG. 5. PROBABLE LOCATION OF ISOTHERMIC LINES IN THE SECTION OF A CONCRETE GIRDER

On Project 12 the deflections were about twice as much as those on Project 220. This would indicate that daily variations in stresses in the steel on this project would be as much as 5,000 lb per sq in. due to change in temperature alone. Unfortunately the strain gages that were used in the second set of readings on this project failed to work. Two trucks, each weighing 12 tons, were placed in various positions on the span, and the deflections due to the load from them are shown in the diagrams. It will be noted that these deflections from 0.016 to 0.020 in., amount to about one-sixth as much as the deflections caused by quick variations in temperature.

On Project 220 deflection of the span due to seasonal changes was noted. While the measurements were being taken, the temperature of the air varied from 30 to 80 F, and the average temperature inside the concrete varied from 35 to as high as 70 F. These gradual changes in temperature, however, caused no rise and fall in the spans.

It is recognized that these tests are not conclusive, as no remedy for the serious stress condition found to exist has been indicated, and no definite information has been acquired as to a method of predicting such stresses. The tests do indicate, however, that very serious stresses exist in unsymmetrical concrete sections, and that considerable further work is needed along the following lines:

1. To determine the rate of radiation and absorption of heat by concrete of various kinds under different conditions.
2. To evaluate the stresses to be anticipated in unsymmetrical sections of given dimensions.
3. To work out a method of offsetting these stresses.

The following general conclusions would seem justified by the work that has been done on these two projects in Georgia:

1. With quick changes in the temperature of the air, such as occur between night and day, there is a marked difference in the temperature of a concrete slab and that of a heavy girder or column.
2. The difference in temperature between the slab and the girder is greater for thin than for relatively thick slabs.
3. The difference in temperature between the slab and the girder or column causes a warping or deflection of the section.
4. The change in shape of the section causes very appreciable stresses in the concrete and steel. The measurements made on 40-ft deck-girder spans indicate an increase in stress of as much as 5,000 lb per sq in. in tension in the steel reinforcing between mid-afternoon and midnight.

It is to be hoped that the results of these tests will stimulate other engineers to pursue similar researches. The outcome may be awaited with interest. In the meantime, the present results will indicate some of the important possibilities and potential dangers.

Greece Engaged on Huge Reclamation Project

Drainage and River Control Works Under Construction by an American Firm

By Ralph H. Chambers

Member American Society of Civil Engineers
Vice-President, The Foundation Company, New York, N.Y.

Faced with the necessity of providing for a million Greek and Armenian refugees expelled from Asia Minor by the Turks in 1922, the Greek Government has been taking vigorous means to provide agricultural land for them within its jurisdiction. West of the City of Salonika lies an area of 600 sq miles of fertile though swampy land, much of which has been subject to inundation by the Axios and Aliakmon rivers and their tributaries. In 1925 the government entered into a contract with an American firm to drain and reclaim these lands. Of the total work, estimated to cost $18,000,000, two-thirds is now complete. The major part of the work—the digging of canals and building of levees—is being accomplished by dredges and large drag-line excavators, although an average of 2,500 men are employed on the project. Earth excavation has been handled at a very low price. The engineering plan for reclaiming the land in the Plain of Salonika is explained by Mr. Chambers in this article.

Greece is a mountainous country; only about one-fifth of its area is tillable. It is, however, an agricultural country and exports large quantities of such products as citrus fruits, olive oil, figs, and currants. It raises wheat, barley, oats, animal and dairy products, and some of the finest tobacco. The country finds it necessary, however, to import a large part of its foodstuffs. Since the destruction of Smyrna by the Turks in 1922, and the expulsion from Asia Minor of the Greek and Armenian colonists, this condition has been gravely accentuated.

At that time the support and rehabilitation of the million refugees who returned to Greece became a critical problem, which was undertaken at first by American charitable societies and later by the Greek authorities through the League of Nations. Within 18 months after the exodus from Asia Minor, the Refugee Settlement Commission of the League of Nations settled more than half a million refugees in new villages in Greece at a cost of about $250 per agricultural family. But the need for farm lands is still acute, and the Greek Government is employing vigorous measures to provide them.

The Plain of Salonika, lying west of the city of Salonika, in the middle of Macedonia, offers an opportunity for the reclamation of large areas of very fertile land. This plain, of comparatively recent origin, has been formed largely by the alluvial deposits of two large rivers, the Axios, formerly the Vardar, which drains a large area of Yugoslavia and flows into this region from the north, and the Aliakmon, coming from the mountains of western and southern Macedonia, which enters the plain from the south. There are also a number of rivers and drainage channels flowing from the north and west which empty their waters and deposit their detritus on the plain.

In the middle of the fourth century B.C., in the time of Philip of Macedon and his son, Alexander the Great, the capital of Macedonia was Pella, which was situated on the high ground in the north central part of what is now the Plain of Salonika. Pella was then on the sea coast, on the Thermaic Gulf of ancient Greece, now the Gulf of Salonika. By the beginning of the Christian era the northern part of the gulf had been cut off by the deposition of silt from the rivers, and except for a narrow channel to the sea, had become landlocked. It was then known as the Loudias Sea. Five hundred years later the Loudias Sea had become an inland lake draining into the gulf through a small and sluggish stream known as the Loudias. In Fig. 1, copied from an old German book, these successive changes are shown. At the present time what was the Loudias Sea is a shallow swamp overgrown with reeds and known as Lake Yenitsa. The site of Pella, originally on the sea coast, is now 19 miles inland.

The Plain of Salonika has an area of about 347,000 acres, the greater part of which is either permanently submerged by lakes and swamps or is subject to inundation by the rivers. North of the main plain are lowlands in the valley of the Axios, west of the present bed of the river. East of the Axios, in the northern part of Macedonia, are the Lakes Ardzan and Amatovo and the lowlands surrounding them. All this country, having an area of about 50,000 acres, is subject to inundation by the Axios. It is very fertile, even the sections that are now submerged. This fact has been proved both by test and by actual use after reclamation.

Floods occur at any time from the first of December to the end of April, but they may and do come even later. The greater number occur in March. The damage done is so great that the cultivators of the plain and of the other low-lying lands are kept in continual fear. The annual precipitation is not definitely known but from information available it appears that it is probably 17 in. at the seacoast and slightly more than 33 in. in the mountains.

The temperature varies widely, and is high during a large part of the year. In December, January, and February it varies from 9 to 75 F; in the early spring and late fall, it ranges between 19 and 86 F; and in the late spring, in the summer, and in the early fall, it varies between 36 and 109 F. The large swamp areas and high temperatures of much of the year favor the propagation of mosquitoes. Malaria is therefore very prevalent.

COMPREHENSIVE PLAN ADOPTED

These were the conditions that existed in the spring of 1925, when the Greek Government decided to reclaim these lands. The object was twofold, not only to secure a large additional area of much needed fertile land but also to put an end to the conditions that had made this region insalubrious and unattractive to settlers. In March of that year a contract for the reclamation works was entered into by the government and The Foundation Company of New York. Work was begun in the following October. The estimated cost was $18,330,000. At the present time about two-thirds of the work has been completed at a cost of $11,500,000.

The project may be considered as divided into five general parts: (1) the training and control of the river

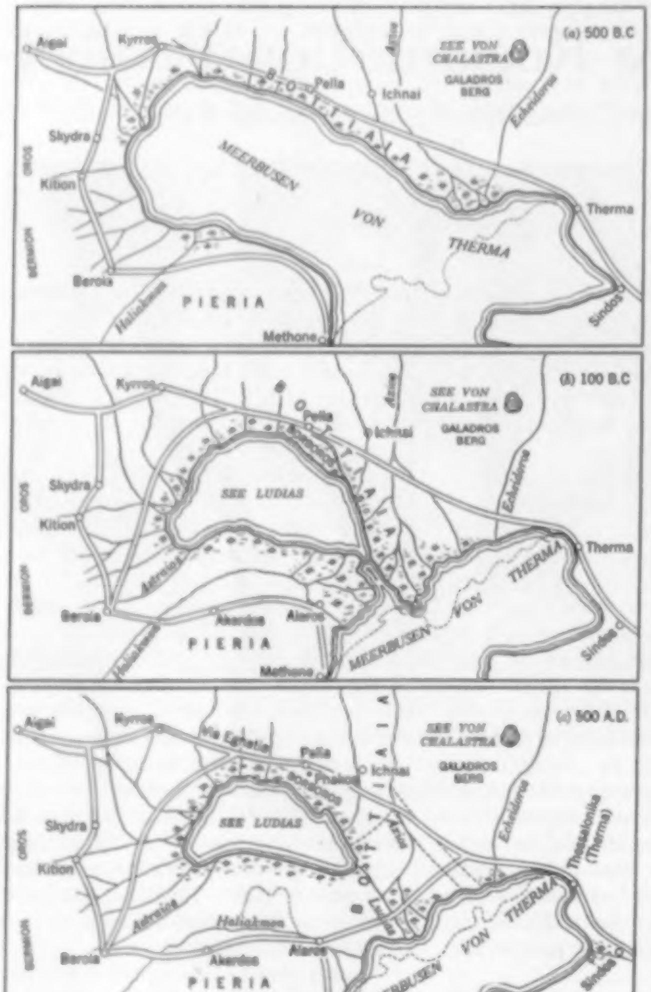


FIG. 1. SUCCESSIVE TOPOGRAPHIC CHANGES IN THE PLAIN OF SALONIKA, MACEDONIA
From an Old German Book

Axios, together with the draining of Lakes Ardzan and Amatovo, the collection and control of the drainage from the highlands west of the river, and the protection of the lowlands in the valley of the Axios and in the plain itself; (2) the control of the river Aliakmon and the protection of the plain from periodic flooding by this river; (3) the collection of the drainage from the hills and mountains west of the plain, by the training and control of the rivers and drainage channels flowing from that region and the disposition of these waters outside the plain; (4) the disposition of the drainage from the hills north of the plain and the draining of Lake Yenitsa and of the swamps along the course of the Loudias; and (5) finally, the diversion of the lower course of the Axios River to prevent the filling up of the harbor of Salonika and the complete landlocking of that city. Reference to the map, Fig. 2, will show the relative locations of the rivers, lakes, and streams mentioned.

RIVER AXIOS GIVEN A NEW MOUTH

The Axios River, which is about 200 miles long, flows for a considerable distance in Yugoslavia and, issuing from the mountain gorges, enters Macedonia from the north. It continues in a southerly direction to the Plain of Salonika, but there it turns to the southeast and empties, through a delta, into the Gulf of Salonika, opposite the city of Salonika. Its average slope is 1 in 2,000.

The principal tributaries of the Axios, from the west, are the Gorgop and Tumba rivers, which discharge into the Axios not far south of the northern frontier of Macedonia, and the Goumenitsa and Gurbes, which flow into it north of the Plain of Salonika. East of the Axios, in the northern part of Macedonia, lie the Lakes Ardzan and Amatovo, into which the Selimli flows, carrying the overflow from Lake Doiran, on the border between Yugoslavia and Macedonia. Far to the south, the Galikos River empties into the Axios near its mouth.

The flow of the Axios, like that of all the streams in this region, is of a very flashy character. During the summer, except when heavy rain storms occur in the mountains, the flow is small, but in times of maximum flood it reaches 133,000 cu ft per sec. The bed of the river, like that of most alluvial streams, has been slowly built up by deposits of silt until in some places the banks are 10 ft above the level of the land in the valley little more than a mile away. The alluvial soil through which the river flows is so light that, during floods, severe erosion of the banks takes place. The river overflows into Lakes Ardzan and Amatovo on the east, into the lowlands in the western part of the valley, and into the Plain of Salonika and Lake Yenitsa on the west. The general plan for the control of the Axios consists of protective embankments to confine the river to a definite, permanent channel. These are located on both sides of the stream along its whole length, except where the ground is higher than the level of maximum floods.

FLOW RECORDS NOT AVAILABLE

Records of the maximum flood discharge of the rivers in this region were not available. The basis of the computations of the maximum discharge of the Axios was a mark showing extreme high water in the mountain gorge of the Yugoslavian frontier, which had been carefully made by the Greek State Railways. The area was obtained from careful cross sections of the river bed made at this point, and the gradient was secured from the high-water marks of a subsequent flood. These data, when applied in Manning's formula, gave a close approximation of the maximum flood discharge. These determinations have been checked by later flood heights and have been found reasonably accurate.

The protective embankments have been placed at an average distance apart of 4,600 ft and have been raised to such a height that a sufficient capacity will be provided for the maximum flood discharge. A maximum percolation slope of 1 in 7 has been the basis of design for the ordinary embankment material (alluvial soil). From this a standard section has been developed having a width on top of 13 ft, a freeboard of $3^1/_4$ ft, a slope of 2 in 1 on the river side and of 3 in 1 on the other, and berms 13 ft wide at a level $8^1/_4$ ft below top, when the embankment exceeds 10 ft in height. Vegetable soil on the site of the embankments has been stripped to a depth of 5 in. and placed on the surface of the embankment as a top dressing. The surface, especially on the water side, has been manured with chemical fertilizer and then sowed with a deep-rooting grass seed obtained from Italy.

Borrow pits have usually been located on the land side at a distance of $16^1/_2$ ft from the toe of the embankment. They are limited to a depth of $3^1/_4$ ft. An undisturbed space 33 ft long has been left between pits 300 ft long to prevent excessive drainage flow near the embankments. Where occasion demands, these pits are dug on the water side of embankments but at a safe

distance away. They are then utilized as drains. When suitable material for embankments is not at hand, it is transported from the nearest satisfactory pits, but in general satisfactory material has been found where needed.

The construction of embankments by drag-line buckets has developed adequate consolidation. A 15 per cent increase in the height of embankments has been made to allow for ultimate shrinkage. Wherever necessary, riprapped road crossings over embankments have been provided. The average distance between the protective embankments on the east and west banks of the Axios is about 4,600 ft. In the upper course of the river there are several sharp bends which might in time induce a cutting of the protective embankments. To prevent this, spurs of riprap in wire netting have been built in the river at these points.

The present mouth of the Axios is 15 miles southeast of its former position at the northern border of the Plain of Salonika. It has been computed that the amount of silt deposited at the mouth of this river is 6,000,000 cu yd annually. Considering this fact and the extent to which such deposition has already encroached on the harbor of Salonika, it is evident that within a comparatively short time the harbor will be filled and the city left inland. It has therefore been determined to divert the river Axios so that from a point about 10 miles above its present mouth it will flow due south and empty into the main gulf at a point about 14 miles west of the city. The gulf there is about 16 ft deep and the sea bed has a slope of 1 on 100. Experiments relating to littoral drift have shown that very little movement of sand or silt that might block the new mouth may be expected. Only strong southerly winds would have an adverse effect, and the prevailing winds are northerly. If necessary, spurs and jetties will be constructed to maintain the position of the new river mouth. Protective embankments will be built on both sides of the new channel, and the channel itself will be excavated by suction dredges.

Diesel-Electric Drag-Line Excavator on Circulatory Canal
With 155-Ft Boom and 5-Cu Yd Bucket

The northern section of the protective embankment on the east bank of the Axios will prevent its flood waters from entering the Lakes Ardzan and Amatovo and inundating the lowlands around them as they formerly did. The lakes have been drained by excavating a channel from them to a point of discharge into the Axios about 8 miles below. This channel, which has a bottom width of 39 ft, runs behind, and parallel to, the protective embankment along the river. It is provided with two controlling sluices, one at the point of discharge into the river and the other near its upper end, at the lakes. The spoil from the lower five miles was used to construct the protective embankment along the Axios.

This drainage channel connects with the main drains that run through the whole length of the middle of the lakes. These drains were excavated by floating equipment to a length of about $12^1/_2$ miles and a bottom depth of 26 ft. As the waters of the lakes receded, tributaries leading to the main drains were excavated. The lakes, which had formerly a maximum depth of 20 ft, have now been completely drained except for a small part, about 1,000 acres, of Lake Ardzan, which will remain as a permanent shallow lake. Because of the additional expense, it has not been deemed advisable to reclaim this area.

The drainage area of the River Galikos lies east of the Plain of Salonika. For the greater part of the year the river is dry, but in times of maximum flood it carries about 25,000 cu ft per sec. The

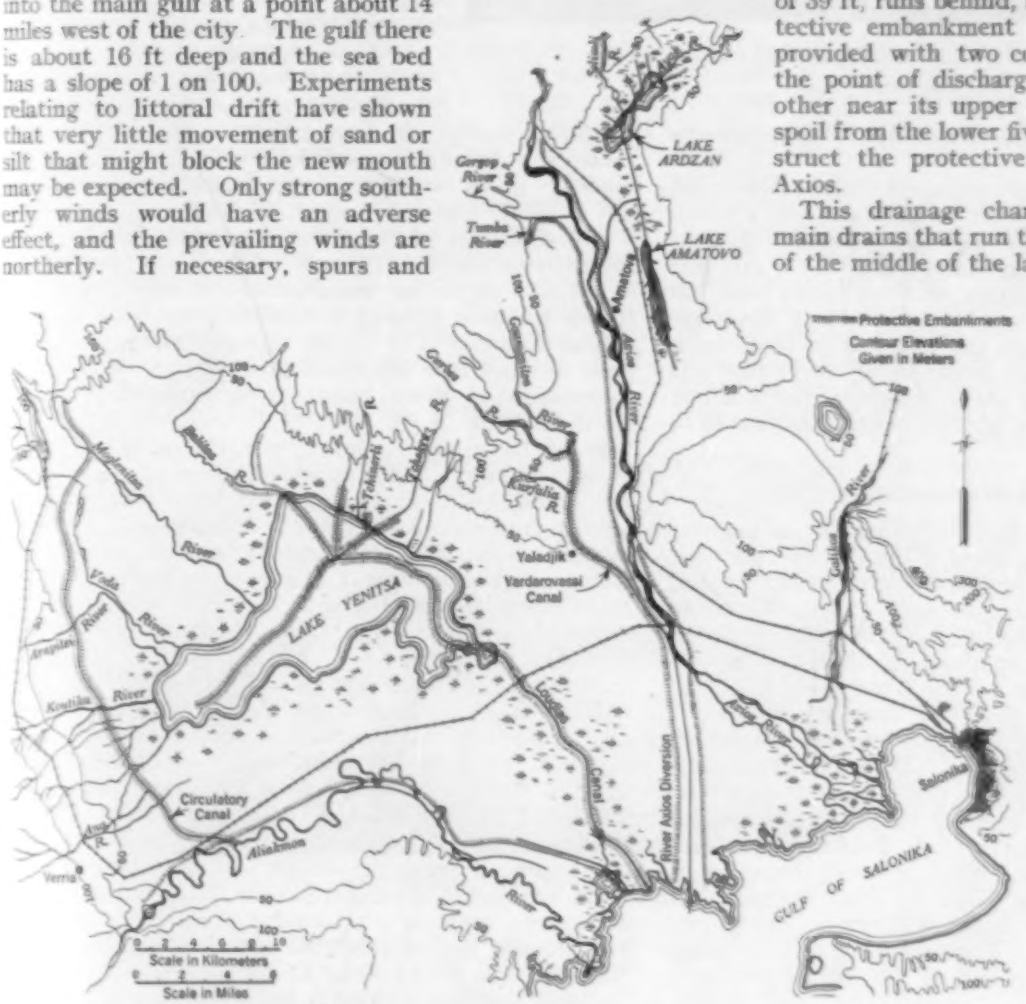


Fig. 2. Reclamation Works on the Plain of Salonika
$18,000,000 Drainage and Flood-Control Project Now Under Construction in Macedonia Under a Contract Between the Greek Government and an American Firm

principal protective works required on this river are the riprapping of the whole bed and banks in the vicinity of a railway bridge near its mouth, where the flood velocity would exceed $4^1/_2$ miles per hr. Such a velocity generally has been adopted, in these works, as the safe maximum. At its mouth the normal bed of the river has been blocked and has been diverted into an old course which discharges to the gulf over waste land near the present delta of the Axios.

The training and control of the tributaries that flow into the Axios from the west have been difficult. It was finally decided to cut a new channel into the Axios for the Gorgop River and to divert the Tumba into the Gorgop, both of these within the protective embankments of the Axios.

MULTIPLE-BUCKET EXCAVATOR, DIESEL-ELECTRIC DRIVE
On Circulatory Canal

The waters of the Goumenitsa and Gurbes rivers and smaller drainage channels will be gathered into a canal, to be known as the Vardarovassi Canal, which will follow the lowest contours of the Axios Valley at a gradient flatter than that of the Axios, until sufficient head has been accumulated to ensure maximum flood discharge into the Axios even when the latter is in high flood. However, the shapes and locations of the Axios and Vardarovassi catchments are so different that a simultaneous maximum flood in both river and canal would be an unlikely coincidence.

ALIAKMON RIVER AND CIRCULATORY CANAL

The Aliakmon, a river about 125 miles long, issues from a gorge at the southwest corner of the Plain of Salonika, follows close to the hills in a very tortuous and winding channel, and empties into the gulf. The capacity of its channel, which is about one-fifth the maximum flood flow, has been computed from data obtained in a manner similar to those for the Axios to be 75,000 cu ft per sec. When the river overflows its channel, it spills over the north bank and flows over the low ground of the plain into Lake Yenitsa. To control this river, it is being trained into a course nearer the hills; its bows and loops are being straightened out; and a protective embankment along its north bank, similar in construction to those along the Axios, is being constructed. This embankment will be about 17 miles long and will be located at a sufficient distance from the river to prevent any future overflow. Protection and training works to prevent dangerous erosion of the banks are being constructed.

The solution of the problem of draining Lake Yenitsa was less obvious. All the drainage from the west and north of the plain runs into the lake From the north there are three streams, the Tchekre, the Tchinerli, and the Balitza. The last mentioned is only 18 miles long and the other two are not over 12 miles long.

From the west no less than five streams or drainage channels empty into the plain: the Ana, the Koutika, the Arapitsa, the Voda, and the very considerable Moglenitsa. The latter is 43 miles long and has a maximum flood discharge of 38,000 cu ft per sec. Of the other four, none is over 25 miles long and none discharges more than 12,000 cu ft per sec, but combined they pour into the plain, in times of maximum flood, 44,000 cu ft per sec. All of them are flashy; the time within which they rise to maximum flood and subside to normal is usually not more than 24 hr.

The final plan adopted is to collect the waters of all five western streams in a canal that will discharge into the Aliakmon River, and through it into the sea. This has been named the Circulatory Canal. It will be about 20 miles long and will be located so as to avoid, as far as possible, the expropriation of land occupied by villages and to interfere as little as possible with existing lines of communication or water supply. It has been designed so as to balance, as nearly as may be, the necessary cut and fill, to keep the depth of cut as uniform as possible, and to induce velocities of flow that will be non-silting and non-scouring. This latter requirement necessitates the construction of certain weirs and regulation works. In determining the flow in this canal, Manning's formula was used, with a coefficient of rugosity of $n = 0.025$, and velocities limited to $v = 0.54d^{0.64}$ where d = depth of flow.

The discharge of the rivers Moglenitsa and Voda, at the extreme northwest corner of the plain, will be collected in a regulating basin located on waste land, and thence will pass through a regulating weir into the Circulatory Canal. The silting capacity of the basin has been estimated as 160,000,000 cu yd, and it has been computed that several hundred years must elapse before it will be filled.

Below the upper regulating weir, the canal will have a bottom width of 125 ft and an average depth of cut of 15 ft. From the material excavated for the canal prism, a protective embankment will be constructed on the west bank of the canal, of the standard section used along the Axios. On the east bank, a protective embankment of extra heavy section will be built, which will have a top width of 20 ft and a freeboard of 4 ft above maximum flood level.

As additional streams come into the canal, the bottom width will be increased until it reaches a maximum of 180 ft, and the distance between embankments will be extended to 400 ft. The entering streams will be trained into the canal by embankments, falls, and weirs. The low velocities at which they will enter will permit their coming in at right angles to the canal prism, at which points proper riprap protection will be provided.

On account of the difference in characteristics of the catchment areas of the several tributary streams, it is extremely improbable that a maximum flood on all these streams would occur at the same time, but nevertheless it has been decided to make the canal adequate to provide for such a contingency. From the studies and observations made, it has been concluded that a severe storm affecting the Ana, Koutika, and Arapitsa, followed

by floods from the Voda and Moglenitsa, might result in a canal discharge of 35,000 cu ft per sec, but this would be improbable and could only occur in an abnormal year.

As a safety valve in case of floods greater than those anticipated, it has been decided to provide a breeching section through the canal embankment. Through this the waters of the Moglenitsa could pass over an area of scrub and brush into its old channel to Lake Yenitsa.

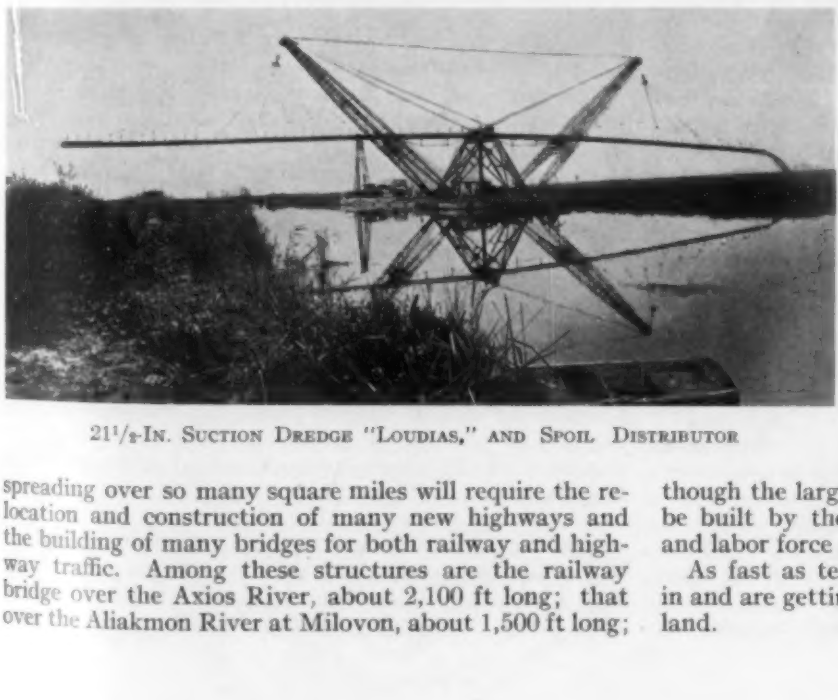
A plan for a similar circulatory canal along the north side of Lake Yenitsa, to collect the waters of the Balitza, the Tchinerli, and the Tchekre, and to carry them into the Axios, proved unsatisfactory because of the large amount of rock excavation that would be required, because of interference with railways and highways and of danger from silt deposits, which would be brought directly into the canal by the northern streams.

DRAINING OF LAKE YENITSA PRESENTED DIFFICULT ENGINEERING PROBLEMS

The ground in the center of Lake Yenitsa is only about $8^1/_2$ ft above sea level, although it is at a distance of 25 miles from the coast. The flow through the outlet must therefore be very slow, and it is evident that to give the canal a sufficient capacity to take the whole flood flow would be economically impossible. It was at first proposed that an impounding basin, covering 3,100 acres, be constructed in the center of the lake, where the flood waters could be stored until they could be discharged through the canal. This basin would not necessarily be flooded every year and could be utilized for pasturage or for growing Indian corn. But for reasons of economy the plan adopted is to use the existing stream known as the Loudias, which, after clearing and excavating, will be known as the Loudias Canal; to limit the capacity of this canal to 5,250 cu ft per sec, which will be sufficient to drain Lake Yenitsa and the swamps adjacent to the canal; and to permit the flood waters to inundate a part of the area now covered by the lake for the short periods that will elapse before they can be discharged through the canal. The total area of Lake Yenitsa is at present about 74,000 acres. It is estimated that, under the conditions proposed, a normal flood will inundate 6,700 acres for 15 hr, and a maximum flood will cover about 15,000 acres for 45 hr.

It is manifest that the construction of hydraulic works spreading over so many square miles will require the relocation and construction of many new highways and the building of many bridges for both railway and highway traffic. Among these structures are the railway bridge over the Axios River, about 2,100 ft long; that over the Aliakmon River at Milovon, about 1,500 ft long; that over the Aliakmon at Kouloura, about 600 ft long; and that over the Vardarovassi Canal, about 550 ft long. Also there are the highway bridge over the Axios River, about 1,900 ft long; the structure over the Vardarovassi Canal; the Verria Road Bridge; the Janchista Road Bridge; the Vestitsa Road Bridge; and the upper Notched Weir Bridge, all between 300 and 400 ft long—and many others of shorter span.

A COMPLETED SECTION OF THE CIRCULATORY CANAL
Reclamation Works on the Plain of Salonika, Macedonia



$21^1/_2$-IN. SUCTION DREDGE "LOUDIAS," AND SPOIL DISTRIBUTOR

PLANT AND EQUIPMENT USED

The construction of these works involves the handling of millions of cubic yards of earth as well as general construction of great volume and almost infinite variety. It may, therefore, be of interest to know that the larger equipment includes 15 revolving drag-line cranes and three clamshell bucket cranes, all having steel booms from 35 to 155 ft in length and buckets ranging in capacity from $^3/_4$ cu yd to 5 cu yd, respectively. In general the cranes are mounted on caterpillars and are driven by steam power, although five of the largest are driven with diesel-electric power. All these machines are of American manufacture.

To show the magnitude of the equipment and the extent of the work to be performed, one of the three largest drag-line excavators is illustrated at work on the Circulatory Canal. In addition to this equipment, a multiple-bucket excavator of German manufacture is being used. It is driven by diesel-electric power, and is mounted on tracks. Some idea of the capacity and performance of the larger of the land machines may be obtained from the fact that under favorable conditions these machines have at times excavated and handled earth at a cost of from $1^1/_2$ to 2 cents per cu yd.

The floating equipment includes a 26-in. suction dredge built in Holland and a $21^1/_2$-in. suction dredge and distributor built in Italy. Miscellaneous steam tugs, lighters, pontoons, tractors, road machinery, and back-fillers complete the mechanical equipment. Although the larger part of these works must of necessity be built by the use of machinery, nevertheless a staff and labor force averaging 2,500 men are required.

As fast as territory is reclaimed, settlers are moving in and are getting satisfactory results from the use of the land.

Distribution of Costs of Assessable Improvements

A Symposium of Practices in American Cities—Difficulties and Possible Remedies

EVERYWHERE communities that are carrying out major street improvement programs are plagued with the problem of collecting the assessments to pay for them in this period of financial readjustment. As a result, a reasonable doubt exists in the minds of many municipal officials as to the sufficiency of the present systems of spreading benefit assessments. The City Planning Division of the Society considered this problem at length during its session at the Annual Meeting in January 1933, at which time a number of papers on the subject were read. One of these, by Hymen Shifrin, M. Am. Soc. C.E., has already appeared in the March issue. In order to further open the subject for the profitable discussion of the membership, the remaining papers are presented in shortened form in this symposium. They relate to the methods in use in the cities of New York and St. Paul, and in the States of Kansas, California, and Illinois. The authors suggest certain modifications in procedure to lighten the burden on the taxpayer and to make the distribution of costs more equitable.

What Is a Benefit Assessment?

By Philip H. Cornick

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IN 1926, I ventured the following definition of the pure benefit assessment:

In its purest form a benefit assessment is neither more nor less than a tax proportioned to the unearned increment accruing to properties as a result of a certain public improvement or public service, the rate of the tax up to 100 per cent of the increment being determined by the ratio between the public expenditure for the purpose, and the aggregate of all the individual increments resulting therefrom.

Since writing this definition, I have found an example that illustrates clearly what I had in mind, that is, the benefit assessment authorized in the statutes of Texas as one of the alternative schemes for financing levee and drainage projects. Under the procedure followed, the separate parcels of land within the area to be benefited are listed; the present value of each is ascertained and set down in one column; and its value after the improvement has been completed is then estimated and set down in a second column. The estimated enhancement in value of each separate parcel is found by subtracting the items in the first column from those in the second, and is entered in a third column. The improvement cannot be undertaken unless the sum of the estimated enhancements exceeds the estimated cost of the improvement. If it does, the governing body acquires authority to proceed, and the statute stipulates that the costs be apportioned among the several parcels within the benefited area in proportion to the estimated enhancement of value received by each.

A RELIEF MAP SHOWING "VALUE TOPOGRAPHY"

Whether or not this exact procedure is followed in assessments for street openings and widenings, every step in it is implicit in any well ordered benefit assessment. Now, in order to visualize precisely what is done whenever a benefit assessment is made, let us start with a base map of the area affected—a map showing the location and dimensions of every parcel of land in the area, which may extend well beyond the boundaries of the city. Let us now erect a relief map on that base, using as the third dimension the present value in dollars of every square foot of land lying within the boundaries of the property lines.

If it is an area such as the city of St. Louis, for example, and if the long narrow canyons of no value represented by the streets are ignored, the "relief map" will show a very broken "value topography." In the downtown business section there will be a high sloping plateau, from which sharp peaks will rise at important intersections. There will also be long ridges of value extending westward from that plateau along the thoroughfares connecting the downtown business section with outlying areas, the ridges themselves varying in elevation at important cross streets, and with existing land uses. Finally, there will be lower lying and sloping plateaus in the outskirts, from which occasional smaller peaks will rise. Such a relief map will show graphically the existing value pattern in the entire region.

The next step will be to estimate the effect of the proposed improvement on that value pattern. The recent major improvements in St. Louis have consisted of widening some of the long streets that lie between the downtown business area and outlying sections, which are still used predominantly for residential purposes. Presumably, the widening will increase the volume of business in the downtown area as well as the desirability of outlying residential areas. Those are the basic assumptions on which the recommendations for the widenings are predicated.

Will the assumed increase in volume of downtown business be drawn wholly from new business due to increase in population, or will part of it be merely a diversion of the existing business that now centers in the outlying shopping districts? Will the stimulus to residential values in the outlying areas be accompanied by an accelerated rate of decline in the nearer, but outmoded, residential sections, which are already in a fair way to become blighted areas? Will the anticipated increase in volume of traffic along the pro-

posed widening divert business from the parallel thoroughfares and thereby depress their values, or will the values along those parallel streets be increased because of the diversion to the newly widened street of the high-speed, through traffic that now interferes with business-producing traffic on the unwidened streets? If the answer to the last question is in the affirmative, how can the increased volume of high-speed, through traffic on the newly widened street be expected to create any new values along it?

I merely ask these questions: I do not pretend to know the answers. Some men, however, have attempted to give answers supported by statistical evidence. On the basis of a study of the effects of the notable improvement on Michigan Avenue in Chicago, Prof. Herbert D. Simpson of Northwestern University states in *The Annals of the American Academy of Political and Social Science*, for March 1930, that "it is doubtful whether large public improvements in the form of streets, highways, and transportation facilities add anything to the aggregate of land values," and that "such improvements will frequently be found to have definitely abstracted land values from other locations."

DIFFICULTIES OF ESTIMATING BENEFITS

But leaving these involved questions out of account, let us assume that we proceed to superimpose on the relief map of existing values merely the increments that we estimate will accrue to certain of the properties on that map. It will be necessary then to establish, first, the boundaries of the area within which we believe distinct and measurable benefits, over and above the general benefits, will fall. That in itself is no easy task. If we adopt the dumbbell shaped district—used extensively in St. Louis and Seattle—it will be necessary next to add to our existing value structure certain estimated increments for every parcel of land within it. These would include the lands lying under the existing elevated plateau and towering peaks in the central business district, the parcels and remnants of parcels lying along the street which it is proposed to widen, and the outlying residential areas extending to, and beyond, the western boundaries of the city.

If a procedure such as this is followed in making the preliminary estimates of benefits, in which tangible, measurable, and weighable indices of the estimates are employed, it will be realized at once that the job is no child's play. The estimates would have to rest largely on guesses, in the making of which many factors would have to be ignored. It would be necessary to ignore the fact that at this or that important intersection, which is so located that it should logically draw to itself a very large increase in the volume of business, the land holdings are tied up in litigation, or are in the hands of notoriously non-progressive estates. Nevertheless, the existence of such properties may distort the "value flow," which might otherwise be reasonably anticipated at any given point, by shifting values to other less favorable locations. Not only the properties directly responsible for this shift would be affected, but also the other properties in the vicinity.

It would also be necessary to ignore the fact that probably only one or two out of a number of otherwise equally important intersections can become thriving retail centers, and that increments of that type will tend to concentrate largely in the lands whose owners are in a position to get priority in adjusting their improvements to the new uses. If they all improve at the same time and in the same manner they may all go bankrupt together, because not enough new business will develop to support them all.

DAMAGES TO PROPERTY AND IMPROVEMENTS

Let us assume, however, that we have done the best we could and have estimated the enhancement which we believe will accrue to every individual parcel and the total of all. The next task will be to estimate the cost of the widening. Here we must take into account not only the value of the lands actually taken, but also damages to buildings and such other damages as the laws and customs of the city countenance. Furthermore, we must estimate not what we think those values and damages ought to be, but what we judge the condemnation commissioners and the courts will decide them to be. After the estimates are made and totaled, they are compared with the total estimated benefits. Finding an ample margin of safety, we recommend the improvement to the governing body, and the project is started.

After the lapse of seven years, or ten years—in some cases even longer—the project is again laid on our doorstep. Meanwhile there has been a buying and selling boom along the street to be widened; the awards for taking and damages are two or more times what we had previously estimated they would be. Going back to our original estimates of benefits, we find that our margin of safety has disappeared. In order to take up the slack, we must not only crowd our assessment levies to the limit of the estimated benefits, but we must also resort to the fiction of general benefit. This general benefit must be stretched to cover not only a part of the sum by which the awards for takings and damages exceed our original estimates, but also all that part of the estimated benefits which falls on properties located outside the jurisdiction of the city making the improvement.

One more major difficulty remains to be considered. Many existing city plans are based on studies of needs made while we were soaring up to, or resting on, that late lamented permanent high level of prosperity. During that period there took place along the lines of some major improvements enhancements in land prices that were truly astounding and that were assumed to be due solely to those improvements. Take the Michigan Avenue improvement in Chicago, for example. In the article by Professor Simpson, previously mentioned, the statement is made that on some of the block fronts abutting on the improvement, the "appreciation of values" amounted to 2,500 per cent. Today, that section of the avenue is lined not only with towering structures but also with equally towering bankruptcies. In other words, the spectacular enhancements seem to have been in prices rather than in values. The owners of such properties are going to wonder where that imputed benefit from the new improvement has gone and where they are to get the money with which to pay their assessments.

POSSIBLE IMPROVEMENTS IN EXISTING PROCEDURE

Now what, if anything, can be done about it? Certain changes in administrative procedure to improve the accuracy not only of the forecasts of benefits, but also of estimates of the costs of land taking and damages, are highly desirable in themselves. They would undoubtedly go far to make possible more accurate forecasts of the economic feasibility of projects under consideration. They would hardly, however, increase the accuracy of the benefit estimates sufficiently to

make them a firm base for financing in a period of declining real estate prices.

There would seem to be only two paths open in the great majority of municipalities during the current depression. The first consists in reducing the improvement program by the elimination of every project that fails to meet the tests of economic feasibility. The second is more restricted in its usefulness. Where constitutions do not stand in the way, and where annual revaluations of taxable property are made on a realistic basis, it would obviate the necessity for predicting, once and for all, the future benefits on which levies would be based. It consists in substituting for the usual type of special assessment, the so-called "special assessment tax," which has been farthest developed in California's Acquisition and Improvement Act of 1925. That act included certain provisions which have paved the way for manifest abuses, and it failed to provide adequate limitations on the amounts or terms of the bonds to be issued under it. These faults, which are quite unrelated to the "special assessment tax" itself, have brought the act into disrepute. The provision here under discussion requires simply that the successive annual installments of the levy on any property be computed by the application to the bare land value of that property, as recorded on the assessment roll for general taxes for the year in question, of a rate determined by dividing the requirements for interest and retirement of district bonds by the aggregate bare land value within the benefited area.

The use of this method obviates the necessity for endeavoring to estimate in advance, future enhancements attributable to an improvement, and nevertheless achieves a flexible and automatic adjustment of individual burdens to such enhancements of value as register themselves on the assessment rolls for general taxes.

Practice Developed in New York City

By HERMAN H. SMITH

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ASSESSMENTS for local improvements are supposed to be determined on the basis of the benefit derived from the improvement. Such benefit may be strictly local and confined to the immediate frontage; it may be somewhat broader and embrace the surrounding property beyond the frontage as well as the frontage; or it may embrace a very wide local area, or even an entire community. The benefit sometimes reaches beyond the confines of the community itself, but that part cannot, as a rule, be assessed and must therefore be assumed by the community which carries out the improvement.

Owing to the many factors involved, the equitable distribution of assessments on the basis of the benefit accruing to various parts of a community does not lend itself to an exact mathematical solution. In order to develop a uniform method of distributing the cost of improvements, it is necessary to make certain assumptions, and these assumptions, to have any value, must be derived from a study of local conditions and established precedents. When well established precedents exist, they must be carefully analyzed in order to arrive at a consistent and equitable policy, and where such precedents do not exist, care must be exercised to avoid results which are unfair and inequitable when compared with those of former practices and policies.

It may be argued that a community which has never resorted to assessments for local improvements would in the end be better off if it refrained entirely from adopting such a policy, but the fact remains that as the calls for expenditures increase, few communities can afford to keep up with their other activities and at the same time meet the demand for such local improvements as the acquisition of streets and parks, the widening of streets, and the construction of sewers and highways and other city planning projects, without resorting to the policy of levying assessments for benefit.

While it may appear logical to assess any piece of property directly for the special benefit it is supposed to receive from an improvement, such benefit, unless clearly reflected as an increase in the value of the property assessed, may be of little cash value to its owner. When several improvements of a comprehensive nature are carried out in the same general locality, the multiplicity of assessments on such properties causes so great a burden as compared with the actual cash value of the property and its enhancement in value due to the improvements, that the property cannot support the assessment. This situation is particularly acute at the present time.

Many comprehensive improvements were not generally sought by a majority of those now called upon to pay the assessment bills. They were fathered by planning commissions, civic organizations, and local government agencies, and perhaps by real estate owners along the line of the improvements, whose property would receive a substantial and immediate enhancement in value as a result of them. The fact that a part of the assessment would fall on many small home owners in the indirect area of benefit, was in many instances lost sight of owing to the desirability of the improvement and the great clamor made by the various agencies mentioned. Those small home owners, now finding themselves beset by the assessments without receiving any direct or substantial benefit from the improvement, are clamoring for relief. It is a very difficult matter to demonstrate the cash value of many such improvements to the owner of the property in the indirect area of assessment, and this is particularly true when he is faced with several such assessments and when under present conditions his property has in many cases decreased in value since the improvement was made, owing to the general decline in real estate values.

ZONE AND BOROUGH ASSESSMENTS

In New York City the distribution of the cost of comprehensive assessable improvements is generally spread through four zones. These zones are: (1) the frontage on the improvement to a depth of 100 ft; (2) the area surrounding the frontage, varying from a

distance of 200 ft to 1,000 ft or more on both sides of the frontage; (3) the borough in which the improvement is made, or, if more than one borough is especially benefited, those boroughs particularly affected; and (4) in the case of the most comprehensive improvements, the city at large.

The borough assessment in New York City is unique, and the reason for its existence is the fact that before consolidation, what is now New York City consisted of several cities and communities. The Borough of Manhattan, formerly the City of New York, a large part of the Borough of Brooklyn, formerly the City of Brooklyn, and other localities as well, under the laws and procedure which existed prior to consolidation, improved a large part of their respective areas by local assessment. After consolidation, in fairness to the developed parts of the boroughs, which had already paid for their improvements locally, it was felt that unless it could be clearly shown that the improvement was of so comprehensive a nature as to benefit the entire city, the particular borough in which the improvement was made should bear that part of the assessment which could not be assessed equitably on the local area.

DISTRIBUTING THE COST OF STREET ACQUISITIONS

In formulating rules for distributing the cost of assessable improvements in New York City, consideration was given to the status under which the separate communities comprising the city were developed prior to consolidation. It was necessary therefore to base such rules in a large measure on the previous assessment practices.

Generally speaking, the street system in New York City is rectangular, the blocks being about 200 ft in their shortest dimension and varying from 600 to 800 ft in their longest dimension. The depth of the lots is generally 100 ft. As a rule, streets 60 ft wide bound the longest side of the blocks, and the shorter side abuts on streets having greater widths. In addition, some streets more than 60 ft wide have been laid out at intervals along the long dimensions of the blocks, paralleling the 60-ft streets.

Streets having widths up to 60 ft are recognized as necessary for the needs of the frontage, and the cost of acquiring them has in general been charged against the frontage to a depth of 100 ft. The wider streets in general have widths varying from 70 to 100 ft and serve as arteries into which the narrower streets feed, or as crosstown arteries when they run parallel to the 60-ft streets.

Owing to the arterial character of the wider streets, it has been assumed that they benefit more property than the immediate adjoining frontage. For the purpose of arriving at a basis for a general rule covering the distribution of the cost of acquiring streets, the following principles have been adopted:

> Streets having a width of 60 ft or less are considered to benefit only the frontage, to a depth of 100 ft; streets having a width of more than 60 ft, and including 80 ft, are considered to benefit the frontage as well as a more extended local area; streets having a width of more than 80 ft, and including 100 ft, are considered to benefit the local area as well as the borough; and streets having a width of more than 100 ft are considered to benefit the local area, the borough, and the city at large.

ASSESSING BUILDING DAMAGE

The necessity that has arisen for widening many existing streets in developed sections has introduced another factor, that of additional burdensome assessments to cover the cost of acquiring existing buildings and improvements. To meet this situation it was found necessary to work into the rule a provision limiting the maximum assessment on the local area to a proportionate amount of the actual value of the land. To this end many computations were made and a great many assessments that had been levied for street widening projects were analyzed from every angle by studying the effect of applying various percentages of relief to such proceedings.

In view of the great discrepancy between the awards made by courts and the assessed valuation, and because it was impracticable to determine in advance the awards for building damage, consequential damage, and other features, it was felt that a final determination could not well be made until the awards and assessments had been announced by the court. It was therefore determined that the value of the land along the frontage to a depth of 100 ft should be based on the value set by the court for the land taken for the street, adjoining the frontage.

On this basis the determination was made that the maximum assessment on the frontage to a depth of 100 ft should be from 10 to 30 per cent of its land value, the percentage varying in accordance with the width of the existing street and the amount of the widening.

LEGAL PROCEDURE IN CONDEMNATION

In New York City, proceedings for acquiring title to property the cost of which is to be assessed, must by law be authorized by the Board of Estimate and Apportionment. This board is the central governing body of the city and consists of the Mayor, the Comptroller, the President of the Board of Alderman, and the five Borough Presidents, all of whom are elective officials. Besides authorizing the proceeding, this board is also charged with the responsibility of fixing, by resolution after a public hearing, the local area of assessment and the distribution of the cost.

The initial determination by the board is based on the recommendation made to it by its chief engineer prior to the fixing of the date for the public hearing. After the proceeding to condemn has been instituted by the court, and until it is finally consummated and confirmed by the court, the Board of Estimate may amend the proceeding, the boundary of the local assessment area, or the distribution of the cost. The court, after the trial of the proceeding, files a tentative decree as to awards and assessments, which becomes available to the public, and which furnishes the basis for applying the rules so that the final distribution of the cost may be determined.

ASSESSING PHYSICAL IMPROVEMENTS

Under the provisions of the New York City charter, the Board of Estimate and Apportionment must also authorize all assessable physical improvements estimated to cost more than $10,000, and must determine the distribution of the cost as between the local area, the borough, and the city, but the board does not determine the extent of the local area of benefit in such proceedings. The local area of benefit is fixed by the Board of Assessors, which also determines the individual assessments within the local area. As it is feasible to estimate accurately the cost of a physical improvement before it is authorized, the percentage of the cost to be allocated to the local area, the borough, and the city is determined at the time the improvement is authorized.

In general, the rules for distributing the cost of grading and paving streets are similar to those described for the acquisition of land, except that they are based entirely

on street width. The cost of sewers has in every case been assessed on the drainage area. The entire cost of the Wards Island Sewage Disposal Plant, now under construction, has been made a city-wide charge, and it is also the intention, in connection with this plant, to have the city at large assume the cost of the intercepting sewers leading to it. The Board of Estimate and Apportionment has established a policy that all extensive sewage treatment plants of this character will in the future be made a city charge. In many of the older sections of the city it has become necessary, owing to the intensive development that has taken place, to rebuild the existing trunk sewers or to construct relief sewers. Heretofore, the cost of constructing these sewers has been assessed on the drainage area, but consideration is now being given to providing some relief in these cases by charging a part of the cost to the borough.

As the distribution of the cost of all assessable improvements involves the placing of the whole or a part of the cost on a local area of benefit, it affects the extent of the assessment to be paid by each individual owner within the local area. No inflexible law and no mathematical rule or formula can be devised that can be safely applied without discretion and judgment to every assessment case. The only safe and fair method of procedure is to establish laws that lay down general principles, to adopt rules that will serve as a measure to apply to each case, and finally, to provide an agency having sufficient discretionary power to eliminate those hardships that may still exist in the assessments as a result of the application of rules and formulas.

Bond Issues Aid Street Work in St. Paul

By GEORGE H. HERROLD

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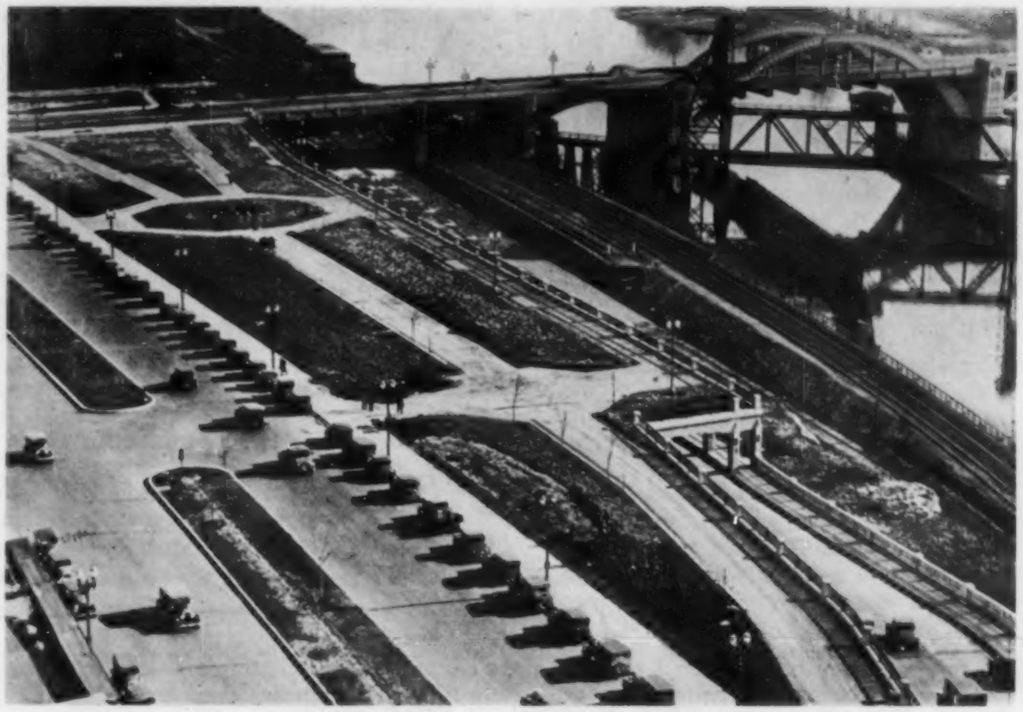
CITY planning determines the way of using land. The principle uses of land in a city are for streets, parks, and building sites. Streets are usually created by dedication in the subdivision plats; the only basis for their width and arrangement was formerly the vision, generosity, or cupidity of the owner. In fact, so little was known about street widths when the majority of our cities were founded that habit and custom probably determined the 60-ft width as suitable to bound two tiers of lots, and the rectangular block system as best adapted to those migrations occurring in all growing cities. Thoroughfare streets were only created in exceptional cases.

From 1870 to 1920, cities worried along with an inadequate and poorly designed street system and improper location of structures and uses. Since 1920 we have been engaged in many experiments. It is obvious that if a street has to be widened it is because sufficient width was not dedicated in the original plat or because the changed uses of property require that the street be widened to bring about an accessibility determined by the use. It would seem basic that the right of way or the width of street should be determined by a consideration of abutting property if the street is for the use of that property.

The principle of benefit assessments is correct. Possibly our method of using this principle requires some refinement. It would seem that some definite standards might be established in regard to city planning projects, and for street widenings these standards would be based on uses of property and the kind of street they needed. For some city planning projects, possibly a way should be found to differentiate between economic needs and social needs.

The principle of benefit assessments was early established in St. Paul. Third Street was widened for eight blocks in 1872. It had a width of $44^1/_2$ ft and was widened variably to between 53 and 57 ft. The pioneer stores had porches as their chief architectural adornment, and the variable width between the new street lines appears to have been determined by the width of the porches or the permanency of the existing buildings.

SOUTH ROBERT STREET GRADE IMPROVEMENT, ST. PAUL



Third Street, When Widened and Improved, Becomes Kellogg Boulevard, St. Paul
Property to River Bluff Acquired and Parked

The entire cost of the proceedings was borne by the abutting owners, the widening being consummated through regular condemnation proceedings. Benefits and damages were made equal.

In 1927 and 1928, Third Street was again widened at a cost of \$1,560,495, by acquiring all the property between it and the river bluff for five blocks. This time a different procedure was followed in determining the benefits. A district including the business streets was laid out, and a graduated assessment was spread, beginning with \$110 per front foot on Third Street and decreasing to \$5 per front foot on Seventh Street. The total assessment was \$598,449. The remainder, \$962,000, was paid from current revenue over a period of about three years. The general benefit to the business district came from the fact that Third Street became the marginal distributor of traffic in and out of that district. St. Paul has recently changed the name of Third Street to Kellogg Boulevard in honor of its well known citizen, former Secretary of State Frank B. Kellogg.

In 1913, Robert Street was widened from 55 to 75 ft by taking 20 ft from the west side of the street for 12 blocks. Robert Street runs at right angles to Third Street and the Robert Street Bridge spans the Mississippi River. The entire cost of the proceedings, approximately \$1,000,000, was made a benefit assessment against a strip of land one-half block in width along each side of the street. In 1923, Robert Street was extended northward to connect with University Avenue by cutting through a hill. This cost \$260,000 and was made a benefit assessment against the same owners who had paid for the original widening. It was considered a completion of that project.

In 1927 a new Robert Street Bridge was built over the Mississippi River at a cost of \$1,800,000 and paid for by a county bond issue. Through a period of three years other widenings were carried out, making Robert Street a continuous thoroughfare for a distance of three miles, to South St. Paul, where it connected with a state trunk highway. The value of the property on Robert Street was never enhanced to equal the assessment of \$1,260,000. Values on this street increased gradually until 1928, when they came to a standstill. It was the desire of the owners in 1913 to make this street an arterial street, and this they accomplished. The widening of the street started values upward, but the increasing automobile travel of a through character pulled values downward.

At the present time, St. Paul is carrying out a \$16,000,000 improvement program over a five-year period. Of this, about \$8,000,000 goes to street widenings and \$500,000 to aid in paving widened streets. The fact that bond money is available for these widenings has made it extremely difficult to assess more than the immediate benefit. That benefit which will accrue from year to year during the next decade, as uses of property and character of buildings are changed and developed to conform to the new street, cannot be established now.

As each project has passed in review before the council, it has tended to decrease the amount of the assessment established by the Bureau of Assessments and Valuations and to increase the amount paid from bonds. This unfortunate tendency has not been retarded by the "depression." Of the street widening projects that have been carried out, from 27 to 33 per cent of the cost of the land and buildings acquired was assessed on the abutting property as a benefit, the cost of the land being from 12 to 48 per cent of the total cost of the improvement.

As regards the paving of streets in St. Paul, there is a charter provision that not more than 24 ft of pavement may be assessed against abutting property, that is, the cost of 12 ft of pavement against each side of the street, and this assessment cannot occur oftener than every 15 years. This width was established on the theory that 24 ft of roadway was sufficient to serve abutting property, and that, if any greater width was required, it was for the benefit of all vehicles, as it facilitated the distribution of commodities and people, and should be paid for by the city at large.

As a means of paying for the additional pavement, St. Paul established a wheelage tax in 1920. Because of proximity to another city that did not have a wheelage tax and because the state legislature limited the amount the city could collect by this tax, it was abandoned after about three years. St. Paul now has an annual fund of \$400,000 as a paving aid raised by a mil tax, which pays for intersections and for pavement width beyond the 24 ft required by the street. Paving aid covers from 44 to 58 per cent of the total cost.

Where land is required to widen a street, the street itself should furnish the land and pay for it, for the simple reason that the value of this land was made by the people who live in the city and not by the owners. The owner receives his remuneration in the increased value of his remaining property and in the accessibility which his business requires.

Building damages can be arrived at accurately, beyond dispute, but the determination of land values is difficult. Expert testimony may develop a difference of opinion that cannot be reconciled. The courts lean toward the owner, whose land, theoretically at least, is being taken from him.

The elimination of land value as a consideration in eminent domain proceedings must be brought about before city planning projects can be carried out on a sound basis. Land values have been created by the people, not by the owner. Why should the citizens at large pay the owner the increment in value which they have created, in order that his property may be made usable and accessible by a suitable street? The only difference between a lot in the Sahara Desert and one on Fifth Avenue, New York, is the spirit of business that surrounds it; and the spirit of business in New York City is created by the people of that city and not by the owner of the lot on Fifth Avenue.

Kansas City's Assessment Plan

By G. G. McCAUSTLAND

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IT has been rightly recognized that in most instances the assessments levied by benefit districts for the cost of public improvements have been largely arbitrary, and that generally the district is made large enough so that the amount of the individual assessment will not be too great. Many schemes and suggestions have been made for lightening the burden on the owner of real estate in sections of a city where public improvements are considered necessary to the welfare of the city as a whole. Among these proposals are: (1) that all improvements be financed entirely by city bond issues; (2) that the amounts to be paid within a benefit district be graded according to the distance of the piece of property or ownership from the improvement; (3) that the benefit districts be enlarged; and (4) that the city institute or increase city automobile license fees or gasoline taxes and use them exclusively for public improvements.

Each of these methods has its objections. The proposal to finance a street widening project entirely by a city bond issue is not equitable since the people living, or owning land, in the vicinity of the improvement will benefit from it to a greater extent than those a considerable distance away. Grading the amounts paid within a benefit district, or enlarging the district, usually results in the inclusion of property to which the benefits are more theoretical than actual. It is humanly impossible to establish the boundary of a benefit district in such a way that it will not include property to which no benefit accrues from the improvement.

Charging the automobile user for making an improvement may seem the logical means of financing until we stop to consider that in many cities, such as Kansas City, the automobile owner already pays a great deal in taxes and fees for the privilege of having a car. He pays city and county personal taxes on his car, a state license fee of no mean amount, a city license tax, a city driver's license, a city and state tax on gasoline, and a government tax on gasoline and oil. If the revenues derived from all these sources could be used for maintaining, widening, and improving the city streets, the tax required on real estate would be small. The large percentage of this income is now used to operate and maintain various governmental departments.

The question has been raised as to the actual benefit to abutting property from street widening. The current opinion seems to be that abutting property derives the most benefit from the improvement, but this is open to serious question. It depends on the relation of the widening to the city's major street plan. If the widening is along a traffic artery connecting outlying districts and a business center, the enhancement of value to abutting property is doubtful, as the tendency of motorists seems to be to make the street a speedway to get to the business center as quickly as possible, with no stops. The people who benefit the most are those near the ends of the project, whether their property abuts on the project or not. If the widening is for the purpose of giving greater access to local business districts, it undoubtedly benefits the abutting property. The distribution of assessments should be different in each of these cases.

In Kansas City a traffic artery was necessary to give access to the business district from a certain area of the city, since the routes of travel then in use were badly congested. The abutting property contributed heavily to the cost of the improvement on the ground that it would increase greatly in value. That was 18 years ago. Today there is still a good deal of that ground vacant or occupied by small stores or filling stations.

DIVISION OF COSTS IN KANSAS CITY PLAN

Up to the time of the adoption of the new charter for Kansas City, the city was divided into eight park districts. The cost of parks or boulevards within each of these districts was paid for entirely by the district. This led to no hardships, as the sections of boulevard were relatively short and the park areas were small. The boulevards proved also to be of direct value to the abutting property, as it has been shown that the value per front foot has been increased by 50 to 100 per cent.

In the so-called Ten-Year Plan for Kansas City, the cost of the traffic arteries included in the plan is divided as follows: the city is to pay one-fourth the estimated cost of the condemnation of property, one-half the estimated cost of the grading, and all the cost of special structures such as bridges, viaducts, or underpasses. The remainder of the cost, exclusive of paving, is assessed against a benefit district, the limits of which are deter-

mined in the usual way. It is the desire of the city to lay out these benefit districts so that they will not overlap. The cost of paving is assessed against abutting property.

The establishment of a metropolitan district around a city would aid materially in equalizing the cost of a major street project. A large majority of the people in the surrounding territory outside the city limits use the streets as much as the people within the corporate limits. In Kansas City, this metropolitan district would have to cover territory in two states, as many persons from Kansas City, Kans., are employed in Missouri, and one of the best residential districts of Kansas City, Mo., lies across the state line in Johnson County, Kans. Most of these people use the streets in the Missouri city, but relatively few residents of Missouri use the streets in Kansas. This would create a complicated problem in adjustment, which would have to be solved before a satisfactory method of procedure for financing street widening projects could be introduced. The collection of automobile license fees or gasoline taxes covering the metropolitan district would be rather difficult.

Suggested Economies

In cities where conditions are similar to those in Kansas City, a combination of certain economies would aid in solving the problem. When a street widening project is decided upon, the city plan commission should have the power to establish a setback or building line, beyond which no building or structure could project except by agreement with the owner that no damages would be claimed for it as a result of the proposed widening. This would eliminate the prospect of paying for new buildings put up after the project was under consideration and planned.

By limiting the use of moneys collected by license fees and gasoline taxes to street repair and improvement, a saving to property owners could be effected. By paying contractors cash instead of tax warrants for their work, a saving ranging up to nearly 50 per cent of present construction costs could be made. The city could issue bonds or certificates and carry the property unable to pay for the improvement.

The establishment of a board of assessors composed of men thoroughly familiar with real estate values in all parts of the city would furnish a more equitable agency for the assessment of benefits and damages. It would tend to eliminate the possibility of a property owner's being allowed an excessive amount for damages. Also, it would permit the making of a more thorough survey and estimate of possible benefits and damages on a project before it was started.

By combining these various procedures, the amount that would have to be assessed against a benefit district would be relatively small and would work hardship on a minimum number of people.

Direct vs. ad Valorem Method in California

By Oliver D. Keese

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Two methods of assessing the cost of public improvements are provided by the statutes of the State of California. The first is a direct assessment levied on individual parcels of land in proportion to their frontage on the improvement or the benefit received. The second, known as the ad valorem method, consists of assessing the lands within a benefited district in proportion to their value, as determined for taxation purposes.

Under the provisions of the Street Improvement Act of 1911, two methods of assessment are set forth. In the frontage plan, each parcel of land abutting or fronting on the improvement is assessed in direct proportion to its frontage, and the cost of the work done at street intersections is spread over the frontage one-half block in each direction from the intersection. Thus there are two rates wherever the improvement crosses street intersections. The procedure for this plan, as very definitely stated in the act, permits of no individual judgment by the engineer who spreads the assessment. Some chance for argument may be eliminated by this plan, but one at all familiar with the business of spreading special assessments will realize that it is utterly unsatisfactory except where the improvement is of the simplest character.

Under the district plan, the law permits the formation by the legislative body of a district to include all benefited lands to be assessed. In both cases, the legislative body may determine that all or any part of the cost of the improvement shall be paid from public funds. The engineer, or other person charged with making the assessment, must determine the amount of benefit that will accrue to each parcel of land within the district and apportion the costs accordingly. This permits the full exercise of individual judgment and ingenuity.

In 1921, a law was adopted which made the procedure of the Street Improvement Act of 1911 available for use in the unincorporated territory of counties

Storm Drain Construction near Beverly Hills, Calif.
Work Carried Out Under the Provisions of the Drainage District Improvement Act of 1919

OPENING OF MORADA PLACE, ALTADENA, CALIF.
Left, Property Taken Under County Street Opening Act of 1923; Right, Improvements Completed Under County Improvement Act of 1921

under the County Board of Supervisors. The County Surveyor is thereby charged with duties correlative with those of a city engineer and street superintendent. The counties also are empowered, by statute, to carry an improvement into or through one or more cities as well as into or through unincorporated territory, providing the legislative bodies concerned consent to the granting of jurisdiction.

Since 1921, assessments aggregating 30 or 40 million dollars worth of street improvements, including sewers, street lights, water mains, opening and widening, as well as pavement and curbs, have been levied by the Los Angeles County Surveyor's office, on the basis of benefits, with surprisingly good results and little opposition. Liberal use is made of corner influence, depth factors, and curves of our own derivation for determining the relative benefits of odd-shaped, curved, and corner lots. Due account is taken of the highest and best use of the property assessed. In this respect, existing zoning ordinances prove extremely valuable. In some instances, the assessed value of property for tax levying purposes has been used, but it is not recommended for general practice.

AD VALOREM ASSESSMENTS

The phrase "ad valorem assessments" means assessments based on the assessed value of land for taxing purposes rather than on benefits, frontage, or any other direct basis. Since 1907, the statutes of California have contained provisions for levying ad valorem assessments for public improvements in counties. By this method, the total cost of a project is divided by the assessed value of the lands within a district formed for the purpose of paying such cost. The assessment is determined by multiplying the assessed value of each parcel of land by the rate per dollar obtained from the division just mentioned, and is included in, and collected with, the general taxes. Obviously, under this scheme, great injustices may occur during the life of the bonds, because parcels of land remotely situated in relation to the actual improvement, which increase in value faster than similar parcels closer to the improvement, may be required to pay more than the latter. Of course this method is no different from that used for collecting taxes to cover bond issues for schools, public buildings, and other general purposes, but the public, banks, and loan companies seem to shy where street improvements are concerned.

In order to overcome, at least in part, some of the objections to this method of assessing, the Drainage District Improvement Act included a provision whereby the benefited district might be divided into zones, each zone including only lands benefited in like measure. From the standpoint of the property owner, each zone becomes a limited district.

In 1925, the State Legislature adopted the Acquisition and Improvement Act of 1925, designed to combine the features of various other acts with the idea that projects, including acquisition of land or right of way as well as improvement, might be combined and simplified in one undertaking. A provision was made for obtaining immediate possession of the necessary right of way by the issuance and sale of bonds to cover the cost of such right of way. The amount for which these bonds are issued is determined by a preliminary estimate of the cost of the land to be condemned and is deposited in court pending the issuance of the interlocutory decree of condemnation. This procedure permits the construction of the proposed improvement without waiting for the conclusion of condemnation suits, thus allowing the completion and use of an improvement many months or even years sooner than would be possible if the construction had to await the final order of possession. The property owner commences paying off the immediate possession bonds with the next tax levy in which they are included after their sale.

An unusual feature of the Acquisition and Improvement Act of 1925 is that the division of the district into zones (if it is to be so divided), and the determination of the percentage of the total cost to be borne by each zone, must be determined prior to the adoption of, and must be included in, the resolution of intention.

There is much merit in this act if it is judiciously used, but otherwise it may result in considerable harm. Its unwise use in many cases for which it is not suited, due to a lack of proper understanding on the part of persons demanding or suggesting its use, has resulted in many inequitable and unjust assessments. In fact, it has been so greatly abused that the legislature undoubtedly will repeal it during the 1933 session.

Among the lessons learned from the experience with the Acquisition and Improvement Act of 1925, probably the following stand out as of greatest interest: (1) The immediate possession proposition may greatly increase costs and create much injustice, unless used with extreme caution. (2) The ad valorem plan of assessment is suitable for comparatively few street improvement projects and only in districts where values may be expected to remain fairly stable or change with some uniformity. (3) The advisability of trying to combine

FOOTHILL BOULEVARD BETWEEN PASADENA AND ARCADIA, CALIF.
Cost of Improvement Assessed Against a Benefit District. Left, March 1932; Right, July 1932

acquisition with improvement in one proceeding is doubtful.

CONCLUSIONS SUMMARIZED

The direct benefit plan of assessing costs of public improvements appears to be the most equitable, if used with due care by persons properly and thoroughly trained. A good working knowledge of the factors which determine the use and value of real estate is essential and extremely helpful in the equitable determination of benefits. Allocations from public funds, such as gasoline and vehicle taxes, should be made in all cases to take care of other than purely local benefit. In spite of the apparent weight of opinion to the contrary, necessary acquisition of right of way should be considered as an essential part of street and highway improvement and should be paid for, at least in part, as in the case of improvement only, from public funds. One of the many reasons for this opinion is the fact that often property is actually damaged to a greater extent than it is benefited by the opening or widening of thoroughfares required to serve general traffic.

In some quarters, the need had long been felt for a law limiting the special assessment burden that might be levied against real property. This feeling culminated in 1931 in the adoption by the California Legislature of the Special Assessment Investigation, Limitation, and Majority Protest Act of 1931. Under this act, before initiating any proceeding for an improvement to be carried out under a special assessment plan, the legislative body must take one of the two following steps: (1) It must determine that the improvement is feasible and will not cause an excessive burden on the lands to be assessed; invite the property owners concerned by postal card notices to express their desires either to have a preliminary report prepared or to proceed without a report; or (2) it must determine to order a preliminary report prepared without first referring to the property owners. The Board of Supervisors of the County of Los Angeles has adopted the general policy of ordering the "debt limit" report in all cases and has appointed a special assessment committee to hold the required preliminary hearings.

If the ratio of the proposed assessment against any parcel exceeds 50 per cent of its true value, or if the aggregate proposed assessment, plus outstanding liability, exceeds 50 per cent of the total true value of all the lands to be assessed, the project must be modified or abandoned. If a majority protest is presented, the project must be abandoned forthwith, and no new proceeding for the same improvement may be commenced within a year. A majority protest may not be overruled, but the limitation may be exceeded if the legislative body determines by a four-fifths vote that the project is feasible and that the property is able to carry the burden. In the experience of the County of Los Angeles, the cost of preparing debt-limit reports varies from 0.52 per cent to 7.67 per cent of the estimated amount of the assessment, the average being about 2.7 per cent.

Assessing Public Benefits in Illinois

By GEORGE C. D. LENTH
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A TAX is a burden arbitrarily imposed by government for its maintenance. In theory a benefit assessment is not a burden, but is the equivalent of compensation for the enhanced value which the property or person assessed derives from a given improvement. Such an assessment may be imposed on property regardless of whether it is contiguous to the improvement or not.

There is but little uniformity in the statutes of the several states with respect to work done under the provisions of special assessment or local benefit laws. In Illinois, the following kinds of work have been done under the Local Improvement Act: condemnation projects; street pavement projects, including two-level streets; sewer projects, including disposal plants; water supply projects, including pumping stations; sidewalks; house drains; water service pipes; and electric lighting projects, including transformer stations. A proj-

ect to construct a multi-level street or subway at an estimated cost of $43,000,000 is now pending in the courts. Of the several types of improvements that have been constructed under the special assessment or local benefit laws, each type is treated differently. Unfortunately no adequate rule of thumb has yet been prepared or suggested for spreading a benefit assessment against private property.

COMPLETE ENGINEERING STUDIES IMPERATIVE

Before a given project is offered for consideration, definite statutory acts must be performed, such as an estimate of cost and, in the larger projects, plans and specifications. In this way the property owners are informed as to the nature, character, and extent of the proposed project, the several phases of which are usually discussed at a public hearing held prior to the enactment by the municipality of an ordinance for the improvement. After the ordinance becomes a law, the municipality petitions a court of record to appoint commissioners for making the necessary report as to assessments of benefits and damages. The property owner has ample opportunity to object and be heard before the court approves the reasonableness and legality of the project and the levy of the cost. It is because of the vast amount of preliminary legal, clerical, and engineering work required that a period of from two to six years elapses before a judgment of confirmation is rendered on many of the larger improvements.

In the case of the improvement of North Michigan Avenue in Chicago, $40,000 was expended on traffic studies before the project was approved. This traffic investigation has been considered one of the most comprehensive ever conducted for an improvement of this character. In the case of the Wacker Drive improvement along the Chicago River, similar traffic investigations were made at a cost of approximately $25,000. Chicago undertakes thorough engineering studies to justify its major improvements. Other municipalities should give the same kind of consideration to projected improvements.

What constitutes benefits to the property assessed is a much debated subject, but it has been held that benefits cannot be predicated on a future improvement not provided for in the ordinance. Also, the court judgment confirming the assessment is not conclusive if the property has been assessed more than the amount of the benefits. If the benefits do not equal the cost of the improvement, then the difference in cost must be paid by general taxation or by a bond issue levied against the whole municipality. The part assessed against the municipality is known as the public benefit, to distinguish it from the private benefit.

In Illinois, public benefits are assessed by the court having the original jurisdiction over the improvement, and the question of public benefits is not reviewable by the State Supreme Court on appeal. If the public benefit exceeds the amount that the municipality is capable of paying, the municipality may dismiss the proceedings. The assessment of a public benefit is made on the theory that the public as a whole is benefited by the entire improvement. Public benefits in large projects in Illinois are paid by means of bond issues, all of which have previously been submitted to the electorate for approval or rejection.

In the Illinois statutes another means is provided to meet the public's share of the cost in municipalities having a population of not more than 500,000. The municipality may provide by ordinance for a levy in addition to the tax for general purposes now authorized by law, this levy not to exceed one mil on the dollar on all taxable property in the municipality. This fund must be used solely for the purpose of paying the amounts assessed against the municipality for public benefits.

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ELEVATED HIGHWAY ON THE OGDEN AVENUE IMPROVEMENT, CHICAGO
The Special Assessment Was Levied Against 79,000 Parcels of Property

CHICAGO'S LONG-RANGE CITY IMPROVEMENT PLAN

Some years ago, Daniel H. Burnham, one of America's pioneer city planners, inaugurated and prepared a plan for future Chicago. This plan has been in execution for the past 20 years under the general supervision of the Chicago Plan Commission, of which Hugh E. Young, M. Am. Soc. C.E., is Chief Engineer. The actual work is under the direction and supervision of the Engineer for the Board of Local Improvements, a branch of the municipal government. The plan contemplates the opening of new diagonal arterial streets, the widening of existing thoroughfares, and the opening and extension of existing streets. The financing of all this work has been conducted under the assessment plan, that is, the municipality as a whole paid its share of the cost as a public benefit, and the property benefited paid its share as a private benefit. During the past 18 years, the total cost of work under that plan has exceeded $135,647,000. Of this total, 58 per cent has been assessed as a public benefit and the remainder levied against private property.

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Reservoir Storage Above Spillway Level

By H. K. Barrows

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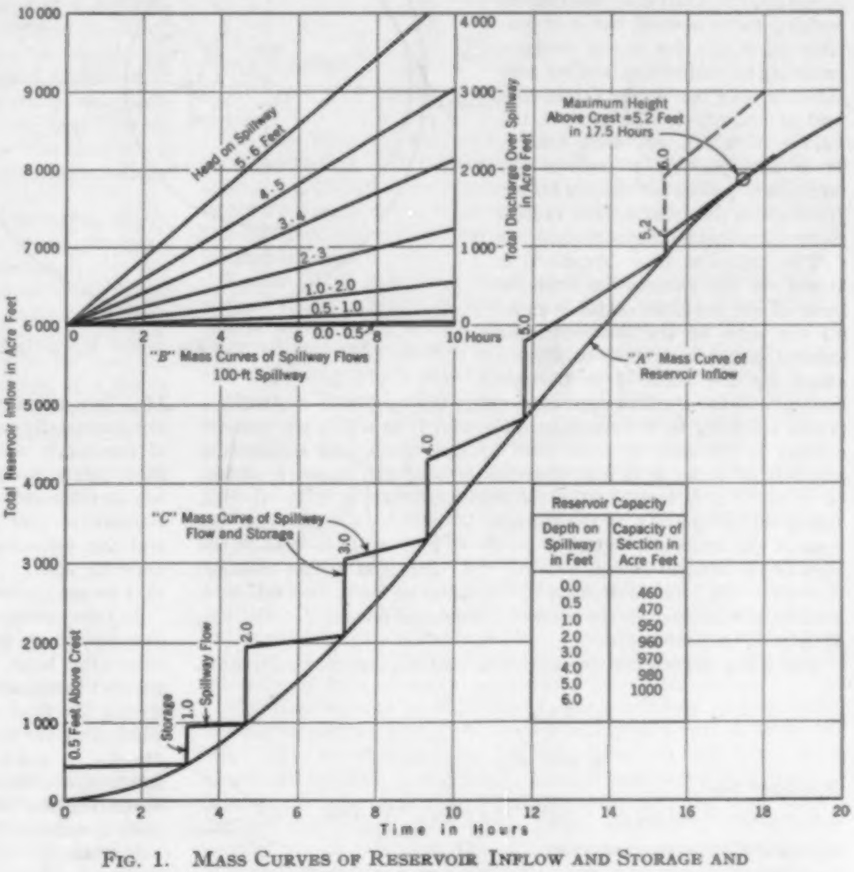
ONE of the problems of importance in flood protection and other hydraulic studies is the effect of reservoir storage above the spillway level in reducing the outflow from the reservoir. Of the many different methods of solving this problem that have been suggested and used, I have found the mass curve method, devised by R. S. Holmgren, M. Am. Soc. C.E., for the 1928–1930 Vermont flood investigations, to be simpler and shorter than any of the methods now in common use. As far as I know this method had never been previously employed. The following explanation of it may be of interest.

In Fig. 1 are plotted, first, a mass curve of reservoir inflow, *A*, prepared from the flood hydrograph, plotting the time in hours as abscissas and the total number of acre-feet of the corresponding inflow as ordinates. On the same diagram is plotted a mass curve of flow, *B*, over the spillway for any convenient period of time, such as 10 hr—long enough to define accurately the slope of these straight-line curves with the same coordinates and scales as for the curve *A*. These curves are computed for rises in the water level above the spillway crest between heights of, say, 0 to 0.5 ft, 0.5 to 1.0 ft, and 1.0 to 2.0 ft., or between other intervals such that the average head on the spillway crest may be assumed without serious error to be the mean of the heights at the beginning and end of the interval. A curve of reservoir capacity above the spillway level (or a corresponding table of such capacities for the intervals assumed in *B*) is shown in Fig. 1.



Depth on Spillway in Feet	Capacity of Section in Acre Feet
0.0	460
0.5	470
1.0	950
2.0	960
3.0	970
4.0	980
5.0	1000
6.0	

Fig. 1. Mass Curves of Reservoir Inflow and Storage and Spillway Discharge

GRAPHIC PROCEDURE TO BE FOLLOWED

Using these data, a mass curve, *C*, of combined spillway flow and storage above the spillway level is drawn, starting at the zero point of curve *A*. The procedure is first to plot the storage capacity for the interval from 0 to 0.5 ft as an ordinate, and from the top of this ordinate to draw a line parallel to the mass curve of spillway flow for this interval on the curve *C*. The point where this last line intersects curve *A* indicates the time and total amount of water taken care of by the rise in level for the first interval, 0 to 0.5 ft. At the point thus found is erected the ordinate of reservoir capacity for the next interval, 0.5 to 1 ft. This value, 470 acre-ft, is taken from the table in Fig. 1. From the top of this ordinate a line is drawn on the curve *C*, parallel to the mass curve of spillway flow, *B*, for this interval, to intersect the curve *A* as before.

This process is repeated until a line parallel to some interval of height becomes tangent, or approximately so, to the curve *A*. At this tangent point and at the corresponding height of water over the spillway, the reservoir inflow is just equal to the outflow. Thereafter outflow will exceed inflow and the water level of the reservoir will begin to fall. The maximum height of water above the spillway level is thus obtained, as well as the time when this occurs, as shown on the time scale at the bottom of the chart, Fig. 1.

SOLUTION ON BASIS OF SUCCESSIVE HEIGHTS OF WATER OVER THE SPILLWAY

The essential advantage of this method lies in the use of mass curves so as to secure a direct solution instead of the usual cut-and-try procedure. This direct solution is made possible by assuming intervals of successive heights on the spillway rather than successive intervals of time, as is commonly done. The simplicity of the method recommends it for more general use.

OUR READERS SAY—

In Comment on Papers, Society Affairs, and Related Professional Interests

Eccentric Circular Sectors

DEAR SIR: I was interested in Mr. Clair's solution of the problem of the areas of eccentric circular sectors, which was published in the "Engineers' Notebook" in the March issue, as I have encountered the same problem in connection with "cubing" of voussoir blocks for arches. The parallelism of the two problems is seen at once if the smaller circle is taken as the intrados curve of the arch and the larger as the extrados curve, and if at the same time the other half of the smaller circle (that not used by Mr. Clair) is considered.

As regards rigorous mathematical accuracy, Mr. Clair's solution leaves nothing to be desired, but it is possible to derive his result without resorting to integration, and by considering only the areas of triangles and of central circular sectors, the values of which are well known. It is also possible to reduce the solution to somewhat simpler terms, from which the troublesome radical expressions have disappeared.

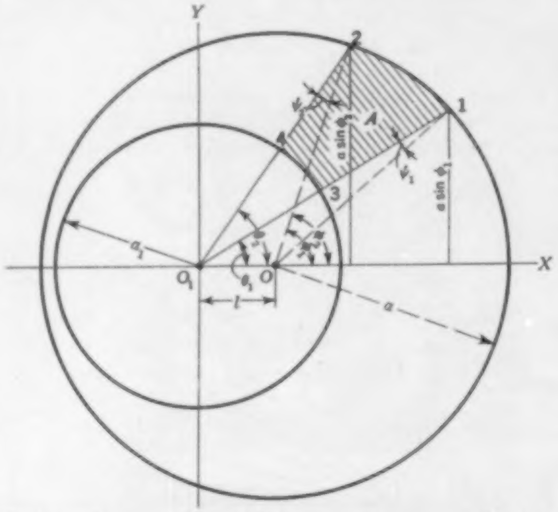


FIG. 1. AREA OF SECTOR BETWEEN ECCENTRIC CIRCLES

The solution here obtained is based on the proposition that the area of an eccentric sector is equal to the area of the corresponding central sector increased or diminished by the area of a triangle having as base the line segment joining the apexes of the sectors. Thus, referring to the accompanying Fig. 1, in which the nomenclature is the same as that used by Mr. Clair, and considering the double areas, it is seen that the area of the eccentric sector, $X - O_1 - 2$, is equal to that of the central sector, $X - O - 2$, increased by the area of the triangle, $2 - O - O_1$. Likewise, the area of the eccentric sector, $X - O_1 - 1$, is equal to that of the area of the central sector, $X - O - 1$, increased by the triangle, $1 - O - O_1$. The difference of the areas of these two eccentric sectors, diminished by the area of the central sector, $3 - O_1 - 4$, is then the required area.

Still using the original nomenclature and writing, for conciseness,

$$\psi = \sin^{-1} \frac{l \sin \theta}{a}$$

$$\phi = \theta + \psi$$

It follows that,

$$2\,A = (a^2\phi_2 + a\,l \sin \phi_2) - (a^2\phi_1 + a\,l \sin \phi_1) - a^2_1(\theta_2 - \theta_1)\ldots[1]$$

or, after a little rearrangement,

$$2\,A = a^2(\phi_2 - \phi_1) - a^2_1(\theta_2 - \theta_1) + a\,l\,(\sin \phi_2 - \sin \phi_1)\ldots\ldots[2]$$

which is the desired expression in fairly simple terms, suitable for solution either by logarithms or by calculating machine.

By substituting in Mr. Clair's solution, as follows:

$$\sin \theta \cos \theta \text{ for } {}^1/_2 \sin 2\,\theta$$

$$a \cos \psi \text{ for } \sqrt{a^2 - l^2 \sin^2 \theta}$$

$$\text{for } l \cos \theta + a \cos \psi$$

$$a \sin \phi \text{ for } \rho \sin \theta$$

that solution is easily reduced to the form in Formula 2.

JOHN L. NAGLE, Assoc. M. Am. Soc. C.E.
Designing Engineer, Arlington Memorial Bridge Commission

Washington, D.C.
March 8, 1933

Combating Traffic Delays

TO THE EDITOR: The type of investigation as to vehicle costs, conducted by Mr. Johannesson and described by him in the March issue, is one on which I have made many observations, computations, and diagrams.

Before many data were available, I adopted 5 cents a mile as the average value of vehicle time for certain highway and city planning problems. This computation was largely based upon the value for a truck, which is about \$3 an hour. As far as passenger vehicles are concerned, this estimate was based upon personal experience and upon the experience of friends, who have missed trains and engagements and suffered other losses because of traffic delays.

In approaching these problems as engineers, we should, perhaps, be conservative and assume a lower value than may be demonstrated to be reasonable or actual. I feel that a value of 1 cent a mile is ultra-conservative, so my tendency has lately been to use a value of about $2^1/_2$ cents.

As to the capacity of highways, which is the other element in the problem, I believe that observations show that the diagrams prepared by Mr. Johannesson provide much larger theoretical figures than are practically possible. I have made hundreds of observations of roadways, which were filled practically to capacity, and they have rarely exceeded 1,000 vehicles per lane per hour. I myself use an estimate of from 700 to 750. However, Mr. Johannesson's estimate of 1,000, or perhaps 2,000, times 1 cent per vehicle minute, and my estimate of 700 times $2^1/_2$ cents, give totals that are not very far apart. In the long run, therefore, the resultant figures that we are both employing are almost identical.

In experiments made in New York City, I had two men drive an automobile on all of the avenues and many of the cross streets hour after hour, almost every day for over a week, counting the number of vehicles that were encountered going in both directions, noting the time they passed selected points on the avenues, and computing the resulting speed at which the vehicles could normally travel. A scatter diagram was then prepared, which showed a much lower capacity than Mr. Johannesson's. However, the scattering was so wide that it confirms one of Mr. Johannesson's points, namely, the importance of the personal driving habits of the motorist. I know, for instance, that I have a tendency to drive at a greater spacing than do most other drivers, so that motorists are apt to try to cut in ahead of me. Such personal driving habits have to be taken into account, and we must get actual statistical information. I have made observations to ascertain the spacing of vehicles on the bridges in New York City, and I have taken photographs from the tops of some of the high buildings to see how the cars travel on Fifth and Park avenues.

To determine the cause of delays in passing through the center of a community, we observed the license numbers of several thousand vehicles and noted the time of their entering and leaving the city limits of Worcester, Mass. Of course, some motorists stopped in the town to do business, but we could very easily identify those who actually passed straight through and measure the time required for this at different hours of the day, with varying amounts of traffic on the streets. We were able to determine the amount of delay caused by congestion, and to ascertain the value of certain detour routes that we were proposing.

Just such information as this must be secured in order to check theoretical computations, and I think that such information has been admirably presented in Mr. Johannesson's paper.

E. P. Goodrich, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
March 6, 1933.

Cost Items Should Be Clarified

Dear Sir: I have read with interest the report on "Contractual Procedure for Foundations of Buildings," by Mr. Viterbo's committee, in the March issue, and feel that it should fill a long-felt want.

I have found that an agreement as to handling extra work based on a fixed percentage is of little value unless the method of determining the "cost" is clearly specified. One large contracting firm with which I have had many dealings sets up an item of extra work in two parts. The first covers the actual cost of labor, to which is added 8½ per cent for insurance, 5 per cent for use of tools, and 10 per cent for overhead, to which total is added the agreed percentage for profit. In looking over a bill thus itemized, the average owner—and a good many engineers too—receive a rude shock when they learn that the expenditure of $1 for extra work really amounts to $1.33. The second figure covers materials and/or subcontract work, which also carries its overhead charge. Clarification of the exact manner of setting up the cost item often saves the engineer from having to make embarrassing explanations to the client.

The lump-sum method, I feel, is much to be preferred where time permits, but the safeguard of a clause in the specifications to the effect that said sum includes all charges is never amiss.

I have never found the unit-price system to be of much value. The bidder usually plays safe by making these sufficiently high to cover any contingency whether large or small, and some contractors give two items under this heading, one to be used for additions and one for deductions. About a year ago, in a list of bids on a million-dollar job, I found a variation of as much as 30 per cent in the unit prices quoted by various reputable bidders.

Fred L. Moore, M. Am. Soc. C.E.
Civil Engineer

East Orange, N.J.
March 8, 1933

Cake Alum for Little Falls, N.J.

To the Editor: In connection with Mr. Armstrong's article on the Montebello filters, in the March issue, I should like to describe briefly the plant for the manufacture of cake alum at Little Falls, N.J.

There isn't anything intricate about making aluminous cake. At first we simply mixed proper proportions of pulverized bauxite and 54-deg sulfuric acid in a tank 4 ft in diameter and 4 ft deep, stirred it until the reaction got under way, and then dumped it on a section of the floor that was boarded off to give a finished cake 4 in. in thickness.

At present we use a fertilizer mixer, 5 by 2 ft, which holds a ton of the mixture. We empty the bags of bauxite and run in the acid simultaneously until a sufficient amount has been added, the thorough agitation causing the reaction to start in a short time. Four of these mixes form a batch for a crystallization pan, where it boils violently, swelling to from 12 to 18 in. thick and spouting up in places 2 or 3 ft above the general level, much vapor being given off at the time. At the start it is red and then it gradually turns gray. By the time it cools it should form a very brittle 4-in. layer that can be readily broken up.

We are hoping to make alum in liquid form some day, because that process saves the cost of crushing and handling the cake. Our present saving, over the commercial sulfate of alumina, results from the elimination of the dissolving of the crude cake, putting the solution through filter presses, recrystallizing, and recrushing.

In computing the cost of alum manufacture at Little Falls, allowance is made for fixed charges on the building as well as the equipment, since a separate building had to be constructed. Counting the fixed charges at 30 cents per ton and the patent rights at 23 cents, the total cost per ton of manufacturing cake alum is estimated at $16.40 as against about $25 for commercial sulfate of alumina.

In making the alum in the winter, we have found that from 15 to 60 lb of soda ash per ton of mix will give a violent reaction, thus heating the mixture to any degree desired. Of course the effect of the soda on the finished product is beneficial, if anything. There seems to be a considerable opportunity for study as to the effect of the addition of larger amounts of soda, thereby producing a mixture of sodium aluminate and alum. The only thing to watch out for is not to get too quick a reaction when coagulating.

In conclusion it might be mentioned that the patents on the cake alum process will expire soon, and therefore such charges will not be a factor much longer.

Frank W. Green
Superintendent of Filtration

Little Falls, N.J.
March 3, 1933

Manufacture of Cake Alum at Kansas City

To the Editor: In connection with James W. Armstrong's very interesting and instructive article on the manufacture of alum at Columbus, Ohio, appearing in the March issue, it may be of interest to compare the experiences of the Kansas City, Mo., Water Department, which manufactures and uses the alum in cake form instead of liquid.

Kansas City's new plant was placed in operation in 1928, with a normal daily capacity of 100 mgd. It is located on the north side of the Missouri River, approximately five miles due north of the city. All materials used in treating the water are delivered on a railroad siding, which runs by the chemical building.

Between five and six million pounds of alum cake are manufactured each year. An average of 1,120 lb of sulfuric acid, 690 lb of bauxite, and 240 lb of water is used in making one ton of cake alum. These quantities are adjusted according to the temperature. In cold weather the acid is diluted to 53 deg Baumé, and in warm weather to 52 deg Baumé. Each car of bauxite is tested and enough bauxite used to deliver a basic product.

Sulfuric acid, 60 deg Baumé at 60 F, is being purchased this year, f.o.b. in tank cars, delivered to our siding at a cost of $15.10 per ton. The acid is unloaded into the storage tanks by applying air pressure directly to the tank car. Extra heavy black pipe is used to carry all acid. The four steel acid-storage tanks, each 6 ft in diameter by 38 ft in length, are made of ⅜-in. steel plate.

Arkansas bauxite is at present being purchased and delivered in bulk in carload lots, f.o.b. on our siding at a cost of $6.25 per ton. Our specifications for bauxite differ slightly from those mentioned by Mr. Armstrong. When it is crushed and dried, it is to contain not more than 4 per cent moisture; not more than 5 per cent Fe_2O_3; and not less than 54 per cent Al_2O_3. No bonus or penalty clause is included in our purchase order. The bauxite is unloaded, crushed to pass a 60-mesh screen, and stored in a heated storage bin, and no difficulties are encountered in maintaining less than 3 per cent moisture. During the fiscal year ending April 30, 1932, the average cost of alum cake delivered to the storage bins was $16.09 per ton.

The acid-dilution and measuring tank is constructed of ¼-in. steel, and is lined with 18-lb chemical lead. It has a capacity of 332 cu ft. An original tank of the same capacity was lined with 10-lb lead. It had been in service about 18 months, giving but very little trouble, when the lead lining began to deteriorate very rapidly, and it was but a short time until the steel shell was pitted beyond repair. A new tank, lined with 18-lb lead, was then installed, and acid leaks started almost immediately. The lining of this tank is spot-welded to the steel shell at 1-ft centers. The temperature studies made by Mr. Armstrong on the dilute acid bear out our belief that the lead lining is ruptured as a result of unequal expansion of the lead and steel. The method in use to locate the breaks in the lead lining is very simple and accurate.

The tank is emptied and the space between the lining and the shell is filled with a soap solution under air pressure. The location of the leak is then determined by the formation of soap bubbles.

To facilitate the removal of alum cake from the crystallizing floor, the floor is whitewashed to prevent bond between the alum cake and the concrete floor. Railroad rails are placed on the floor about 8 ft apart, and the alum is poured around them. When the cake cools and solidifies, a 10-ton chain block is hooked to the rail, and the cake is raised from the floor and broken with a sledge hammer.

T. D. SAMUEL, JR., M. Am. Soc. C.E.
Chief Engineer and Superintendent
City Water Department

Kansas City, Mo.
March 4, 1933

Grazing Exerts Only Minor Effect on Erosion

TO THE EDITOR: Professor Chapman, in Civil Engineering for February, cites examples of recent erosion, asserts as "commonly accepted" that "the only operative cause affecting the huge non-forested area of the intermountain region is overgrazing," and concludes that "control of soil demands control of grazing."

One may well agree with him that erosion continues operative, that in places it is temporarily accelerated, and that control of grazing is desirable. The assumption that the only operative cause of erosion in the non-forested intermountain region is overgrazing is certainly not commonly accepted by those whose business lies upon the open public domain, nor is its corollary that grazing should be controlled in order to control soil a principle on which there is agreement. Erosion is accomplished by weathering and transportation. In general, weathering is far in excess of transportation, and this is particularly true of the so-called intermountain region, where vast quantities of disintegrated and disaggregated material await only the transporting power of water to initiate erosion.

This power, in an arid region, is supplied intermittently by torrential downpours of rain which, as Professor Chapman says, may come only once in 20 years or even less frequently. This transporting force is the principal operative cause of erosion. Its extreme variability results in cyclic and sporadic acceleration and retardation of erosion, in alternation from place to place and from time to time in the same place between degradation and aggradation. Vegetation serves to accelerate weathering and, when sufficiently dense, to retard transportation. The arid region as a whole has ever been characterized by sparse vegetation, subject to considerable variation in density under the vagaries of climate. When compared with the powerful and erratic influence of torrential rainfall, vegetation, with or without grazing, must be viewed as relatively constant and insignificant in the arid region in its effect on erosion.

But grazing is only one of many influences of man on erosion; other influences are shown in the accompanying photographs. Compared with the forces of nature, however, the aggregate of man's influences is puny and insignificant. The Colorado River carries through the Grand Canyon a yearly average of about 260 million tons of material in suspension and 15 million tons in solution, far more in a single year than man has accomplished in a period of years in excavating 240 million tons on the Panama Canal. By building the Hoover Dam man can delay somewhat the progress of eroded material to the Gulf of California, but is helpless to prevent its origin or its ultimate destiny.

Can the sediment of the Colorado, or any substantial part thereof, reasonably be attributed to the activities of the scattering number of stock grazed on its watershed? Since its discovery in the sixteenth century, the Colorado has been recorded as a heavily mud-laden stream, and there is no evidence that its load is more or less now than formerly. If climate permitted, it is reasonable to believe that increasing the density of vegetation on the watershed to that of the humid regions would materially slow down erosion by suspension and would materially accelerate erosion by solution. But climate forbids. If man is reasonably careful to manage the lands of the watershed so as to produce and harvest in perpetuity the normal crop of timber, posts, and forage and, where feasible, to produce cultivated crops by dry farming or irrigation, he will have done his full duty in the matter of control of soil as an incident or by-product of commercial production operations.

Control of grazing in the interests of the stock industry is highly desirable. Any control of soil thereby accomplished should be considered as a subsidiary benefit.

N. C. GROVER, M. Am. Soc. C.E.
Chief Hydraulic Engineer
U.S. Geological Survey

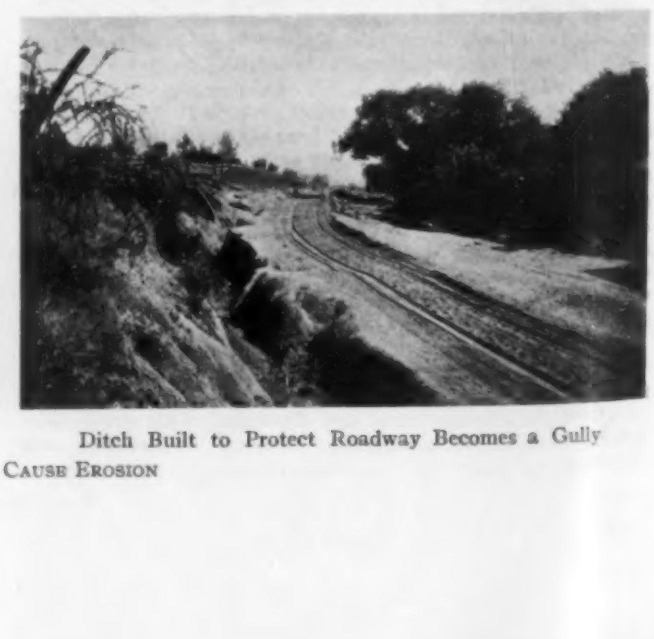
Washington, D.C.
March 8, 1933

Surveying as Relief Work

DEAR SIR: Recent papers and comments in your columns have urged the value of advance planning and basic community surveys in these days. These possibilities cannot be emphasized too strongly.

While few engineers in America today are totally uninformed as to the value of good basic surveys and maps on which to make plans for engineering structures, it may be claimed that many engineers are unacquainted with what really constitutes a good engineering map. In the past, America's faith in the engineer has been, if anything, too profound. It has accepted wonderful achievements from him as matters of course, often denying him some fundamental form of assistance simply because he has heretofore managed to get along without it, yet has bestowed upon his hampered work words of well deserved praise. Nowhere have the handicaps been more manifest than in the lack of good maps—a need that has been felt not only in city and county planning, but in studies for engineering plans of state-wide magnitude.

Leaky Irrigation Ditch in California Causes Hillside Erosion



Ditch Built to Protect Roadway Becomes a Gully

WORKS OF MAN CAUSE EROSION

Recognizing existing conditions in the country at large some years ago, Congress passed the Temple Act, under which the work of expediting the control surveys and the basic topographic maps of the country was given a definite status on the national program. The Temple Act further provided that such civil divisions as states and counties, which might desire to secure priority in the mapping of their areas, could do so by contributing to the expenses thereof. The State of North Carolina is taking advantage of this arrangement, and the U.S. Coast and Geodetic Survey is, at the present time, executing a triangulation and leveling survey of its area.

No branch of engineering offers greater possibilities for unemployment relief than does surveying, for nowhere else is a greater proportion of the money expended paid for labor. This usually runs to something like 75 per cent and may often come closer to 95 per cent. Moreover, one trained engineer may organize a party of three, four, or even more untrained men and, with a little training, execute first-class, economically produced surveys. He may employ rodmen, chainmen, axmen, and laborers for mixing concrete to be used in setting survey marks and, by having two such groups and using each group during alternate weeks, spread even further the benefits of the work he is doing.

Even though trained in other lines of engineering work than surveying, the engineer will have little difficulty in the execution of such basic surveys as triangulation, traverse, and levels, if he will adopt the standards of accuracy and follow the methods of procedure that have been developed by the Coast and Geodetic Survey. These standards and methods are available in the form of manuals prepared and published primarily for the guidance of its own engineers.

R. S. Patton, M. Am. Soc. C.E.
Director, U.S. Coast and Geodetic Survey

Washington, D.C.
March 7, 1933

Engineers Can Relieve the Depression

To the Editor: I read with much interest the three papers in the symposium, "Economics in the Depression," in the December issue, after hearing the papers read at the Atlantic City Meeting. The paper by Mr. Hogan reveals the fact that the introduction of self-liquidating public works stresses a concept that is very likely to be more generally accepted throughout America as time goes on. At present, the Government is the largest single purchaser of construction in the nation. Municipalities are encouraged by means of Federal aid to purchase whatever they can at present low-market prices. The result is wealth created through employment, instead of relief distributed through the unemployment dole. It means that taxpayers are paying for projects at a time when many construction materials are at pre-war prices, and when others are relatively cheap and labor is highly efficient. It would certainly seem sound financing for every community to make whatever improvements it can now rather than to penalize its taxpayers later by purchasing in high-price markets.

Although Mr. Pirnie's article is particularly applicable to waterworks projects, it also seems of interest in connection with sewer projects, which are in essence appurtenant structures. The introduction of new legislation urging the construction of sewer and disposal systems by payments of service rates measured through water meters seems logical. The installation of water works throughout the country is a modern necessity and not a luxury.

The program outlined by Mr. McDonald is a fine piece of logical analysis of the need for cyclical stabilization. His term, "decade book-balance," is certainly different from the usual "annual book-balance" of corporations, and should have a far-reaching effect. It would seem about time for American business to profit by the lessons of the 13 booms and depressions that have occurred during the past 150 years.

One of Mr. McDonald's suggestions is that all corporations become limited dividend corporations until they acquire a surplus equal to their capital, or perhaps one-half of it. This surplus would doubtless have to be tax exempt in the respective states. To be properly liquid such reserve funds should be invested in bonds of the U. S. Government, the states, or political subdivisions thereof. Such bond issues for public works should hereafter have proper serial or sinking-fund requirements. The demand for such securities would tend to reduce interest rates and therefore public taxes. The legislative requirement for these funds might be such that only a 20 per cent depletion could be used in any one year, to be prorated equitably for dividends to stockholders and wages to employees. Inadvertently, wage earners are becoming stockholders in many companies in increasing number through private or market operations.

In order to find a market for such reserve funds in times of economic depression, it might be necessary to confine them to Federal or municipal issues, carrying note-issuing provisions such as are now appended to the Federal Home Loan Bank Bill that applies to Federal bonds up to $3^1/_8$ per cent. This provision gives such bonds note-issuing power through the Federal Reserve Board.

To my mind, however, the most important thing is the fact that civil engineers are finally taking the proper interest in the sound and economical financing of American industry. Perhaps this period of unemployment gives them time to think and to obtain proper perspectives. The analytical mind of the engineer can help solve the financial problems of the nation, as in the past it has helped solve its structural problems. Both problems are linked together. By analysis of cyclical graphs and the adaptation of the knowledge thus obtained, it should, and must, be possible to prevent booms and the depressions that inevitably follow them.

A. P. Greensfelder, M. Am. Soc. C.E.
President, Fruin-Colnon Contracting Company

St. Louis, Mo.
February 28, 1933

Deck Participation with Concrete Arches Obtained Analytically

Dear Sir: In connection with Professor Finlay's paper in the November issue, on deck participation in arches, the possibility of investigation by analytical rather than model methods should not be overlooked. The influence of the deck has been pointed out, long ago, by common sense and theory. The analytical method of determining the extent of it may be given in general outline as follows:

Consider that part of the deck, corresponding to the thickness of the arch ring under investigation, as a beam resting on elastic supports, and loaded vertically on the line of, say, the first column on the left, with the unit load. Draw the deformed axis of the beam for this loading. Afterward, imagine the arch loaded with the same load, l, at the same vertical point, and draw the deformed axis of the arch for this hypothesis of loading. By the theorem of Maxwell, and by imposing the condition that arch and beam shall have the same deflection at the common point considered, one is enabled to write the corresponding equation of equilibrium.

This operation is to be repeated as many times as there are supports, each time obtaining the corresponding equation of equilibrium for the point under investigation. A system of simultaneous equations will be thus secured that will give the reactions acting on the arch as well as those acting on the deck. Once these values have been obtained, the analysis of the arch, as well as that of the beam, can be carried out separately.

It is important to note that, in the case of a symmetrical structure, there is no necessity for drawing the deformed axis of the beam for each hypothesis of loading; the drawing of that axis for the first hypothesis is sufficient, for from that the other deformed axes can be derived. Such procedure will shorten very materially the analysis of the structure.

The use of models, for the purpose of investigation, is desirable in some cases; but their use for designing is not advisable and should be discouraged. Generally speaking, the use of models for investigation is not new. It is said that Galileo, in his attempt to find a formula for deflection, used a model; yet he must have been misled by it, for his formula was not correct.

Alfredo C. Janni, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
March 10, 1933

SOCIETY AFFAIRS

Official and Semi-Official

Year Book for 1933 to Appear in New Guise

YEARLY, in the early part of April, members are accustomed to look for the *Year Book* of the Society, corrected to March 1. In this year of 1933 they will not be disappointed, but they must not expect this volume to appear in its usual garb. Inwardly the *Year Book* will follow the style and general set-up of previous years, and outwardly it will have the same general color, the distinctive tan that is used in regular rotation to avoid confusion with preceding and succeeding numbers. On closer view, however, it will be noted that the cover will designate this *Year Book* as actually Part 2 of the April number of PROCEEDINGS. Under this plan the *Year Book* will go out in the same envelope with Part 1, or the regular number, of PROCEEDINGS. This combination of two formerly completely separate publications has certain basic advantages, notably as regards the postal classification, which is thereby changed with the result that a considerable saving in mailing charges is effected.

Together, the two issues will receive a somewhat wider distribution than a normal issue of PROCEEDINGS. This is because some names temporarily removed from the regular mailing list have been added for April in order that these members might have the privilege of possessing the *Year Book* and the technical information contained in PROCEEDINGS, Part 1.

It is hardly likely that any confusion will arise from this combination of hitherto separate volumes. The colors of the two will be distinctive; and the difference in size will be quite noticeable. It is hoped also that this advance notice will serve to call wide attention to the impending change. Members will await the delivery of the April PROCEEDINGS with more than the usual anticipation. It is expected that every number will be in the mail on April 15, 1933.

Sixty-Third Annual Convention in Chicago

THE LAST WEEK in June is Engineers' Week at the Century of Progress Exposition. Opening on Tuesday, June 27, the Sixty-Third Annual Convention of the Society is to assemble in Chicago, during this and the following three days. Not only will the four Founder Societies have their meetings there during the week, but 14 or 15 other engineering societies are planning to convene at the same time.

Chicago members of the Society, under the chairmanship of W. W. DeBerard, president of the Illinois Section, have formed committees to handle the detailed arrangements for the Annual Convention. Plans are developing rapidly and some of them are completed and can be announced at this time for the information of the many members who wish to take advantage of the exceptional opportunities afforded by attendance at the Convention. The Palmer House has been selected as the Convention headquarters and all the sessions will be held there.

The Annual Convention of the Society will open on Tuesday, June 27, the morning being devoted to addresses of welcome and the President's annual address, followed in the afternoon by a general meeting under the auspices of the Engineering-Economics and Finance Division. In the evening, there will be a joint meet-

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BIRD'S-EYE VIEW OF THE CENTURY OF PROGRESS EXPOSITION

Society's Annual Convention Program Devotes an Entire Day to This Exposition

ing of all engineers under the auspices of the Engineering Section of the American Academy for the Advancement of Science, featuring addresses on the industrial developments of the world during the past century. Wednesday, June 28, has been designated as Engineers' Day at the Exposition, the entire day to be spent on the grounds, observing those sights of especial interest to technical men. For Thursday and Friday, the sessions of the Technical Divisions of the Society are scheduled, some of them to be held jointly with similar groups from the American Society of Mechanical Engineers.

Although the Convention closes on Friday, June 30, it is expected that many will delay their return home in order to complete their sight-seeing at the Century of Progress Exposition. An exceptional opportunity for combining attendance at the meeting with a visit to the Exposition is afforded by reduced railroad fares from all parts of the country. The large and distinguished group of engineers that will be gathered together in Chicago during the week of the Society's Convention will present opportunities for both professional advancement and social contacts. Further details will appear in succeeding issues of CIVIL ENGINEERING, and the complete program will be published in the June number.

March Meeting of the Society

IN ACCORDANCE with the requirements of the Constitution of the Society, the regular meeting for March was held on March 15, 1933, at the Engineering Societies Building in New York. Vice-President Tuttle presided. There being no new or unfinished business to come before the meeting, it was adjourned after brief announcements by the Secretary. The regular monthly meeting of the Metropolitan Section followed immediately thereafter.

Preparation of Memoirs

IT IS THE POLICY of the Society to publish and distribute memoirs of all its deceased members. This practice is based primarily on the conviction that the member deserves such recognition, and further, on the belief that the high estimate placed on the memory of a departed member is only part and parcel of the regard in which he is held while alive. Personal pride in Society membership cannot rise higher than in paying attention and sincere respect to deceased members.

Obtaining a satisfactory form of biographical sketch is not always easy; in fact there are many instances on record, even in the case of the most widely known members and former Society officers, where such memoirs could not be obtained. Death notices are received at Headquarters from the technical and daily press, from friends or colleagues, o. more often from Local Sections. It then becomes a serious problem how to find the best source for the memoir. Here again the Local Section plays an important part, as it usually appoints a committee or assigns the duty to one of its members, who then cooperates with Society Headquarters.

As another possibility, a study of the application of the deceased member for admission to the Society often gives a hint as to those who are most familiar with his life and accomplishments. Such men may be suitable authors of the memoir. Still another possibility is that a member of the family, or a personal friend of the deceased, not an engineer or member of the Society, will undertake the preparation of the memoir.

A most awkward situation arises when two independent sources for a memoir are working without each other's knowledge. This procedure causes obvious embarrassment aside from the fact that it results in duplication of extensive work. Instead, the Society always attempts to bring the interested parties together in the preparation of a single manuscript.

For these reasons it seems desirable that all information or inquiry regarding deceased members and their memoirs should pass through Headquarters as a clearing house. Details to be included in the memoirs or suggestions as to suitable authors are equally useful. The Local Section may be depended on to offer this cooperation.

This hint is given merely in order that interested members may understand the difficulties and therefore be more willing to offer any advice or information in their power. The annual publication of memoirs in TRANSACTIONS is one of the finest accomplishments of the Society. Appreciation of this difficult task is particularly keen on the part of the family of the deceased. It is a meritorious effort, and any assistance on the part of the general membership is decidedly worth while.

United Engineering Trustees, Inc., Makes 1932 Report

UNITED ENGINEERING TRUSTEES, INC., conducts the business committed to it by its Founder Societies under its charter, the Founders' Agreement, the Library Agreement, and the deeds of gift or the bequests for trust funds. Ten monthly meetings are held and the business is conducted through the Finance Committee, the House Committee, and the officers, staff, and special committees. A draft of a budget of revenue and expenditures for the ensuing calendar year is presented each September to the Founder Societies for the information and comment of their governing bodies. After revision, the budget is adopted by the incoming Board of Trustees at its meeting in January. At the same time reports for the preceding calendar year by the President, the Accountant, and the Finance Committee are sent to the office of each Founder Society. Reports are made also by the Library Board and The Engineering Foundation.

The Engineering Societies Building, at 33 West 39th Street, New York, N.Y., is administered on a cooperative basis and not for profit. Offices not used by the Founder Societies and their joint organizations are allotted to associate societies. When meeting halls are not in use by the Founder Societies, other patrons use them and the revenue from these sources reduces the burden for maintenance, operation, and fixed charges.

The Board of Trustees administers trust funds for The Engineering Foundation, the Library, and other purposes. In financial operations and management of property, the trustees have the services of trust companies as custodians of securities, of advisers in making investments, of legal counsel, certified public accountants (as auditors), and a consulting architect.

PRESIDENT ISSUES ANNUAL REPORT

In his report, H. A. Kidder, President of the United Engineering Trustees, Inc., during 1932, said:

"Assets for which the corporation is responsible (real estate at cost, funds at book value, and Library as appraised) total nearly four million dollars. All departments closed the year without deficits. The income of the Administrative Department was $2,329.60 less than predicted in the budget, and the expenditures for all purposes were $18,295.11 less than the budget. This latter amount included, however, a reduced allotment of $6,000 to the Depreciation and Renewal Fund, held in suspense, and a provision of $4,000 for operating cash to start the new year. There was thus a net saving of $8,295.11 as part of the financial program for the period 1 July 1932 to 31 December 1933, adopted by the Board of Trustees in June 1932. The General Reserve Fund was $6,065.60 less than the established amount, $10,000, and the Depreciation and Renewal Fund was $12,000 less than it should have been 31 December 1932; both should be restored from general income as soon as practicable.

"The total book value of all the funds for which United Engineering Trustees, Inc., is responsible is $1,394,355.40. The market value as of 31 December 1932 was $1,102,741.88. The cumulated shrinkage of that date was $291,613.52, or 21 per cent."

NEW OFFICERS ELECTED

Officers for 1933 have been elected. The new president is Harold V. Coes; the first vice-president is Charles A. Mead, M. Am. Soc. C.E.; and the second vice-president is R. M. Roosevelt. The treasurer for 1932, Clifford P. Hunt, was reelected for 1933, while Arthur S. Tuttle, M. Am. Soc. C.E., was chosen assistant treasurer. The Director of The Engineering Foundation, Alfred D. Flinn, M. Am. Soc. C.E., will continue to serve as secretary.

A Preview of Proceedings

In the April number of PROCEEDINGS *will be found three main papers. The first concerns a hydraulic-fill dam constructed on pervious glacial drift; the second deals with the design and construction of a continuous-span railroad bridge of record-breaking size; and the third treats of an improved method of predicting the flow through hydraulic turbines.*

HIGH DAMS ON PERVIOUS GLACIAL DRIFT

THE PROBLEMS connected with the construction of dams on other than solid rock foundations is the subject of a paper by Edward M. Burd, M. Am. Soc. C.E. The paper deals with the general subject of high dams on pervious glacial drift, and Mr. Burd draws liberally from experience gained in the construction of the Hardy Dam on the Muskegon River in Michigan. This dam was placed in operation at full head on April 30, 1931. The embankment, which is 120 ft high, is composed mostly of sand and has a central flexible concrete core wall. Three penstocks, 14 ft in diameter, extend through the base of the deep section.

Of particular geological interest is the part of Mr. Burd's paper in which a description is given of a belt of land across the eastern part of the border between the United States and Canada, in Michigan. While many regions of the earth are devoid of accessible rock foundations, this particular belt is composed of material that has peculiarities of its own. The glacial drift extends to depths as great as 1,500 ft, and the deposit is heterogeneous to such an extent that conclusions from borings and other soil explorations must be averaged in order to establish a workable criterion for judging any dam site.

About two pages of tabulated material are included in the paper, comprising a selection of the most important settlement observations made during the construction of the Hardy Dam. The author also treats such general subjects as the design and settlement of embankments and the design of foundation structures. The conclusions offered at the end of the paper are that:

1. Coarse sand and gravel or river-bottom formations of glacial "mud-stone" will support loads satisfactorily if they are added gradually to a maximum of 13,000 lb per sq ft, with an average allowable settlement of 1 in. per 5,000 lb of loading.
2. Experience with this type of foundation indicates the probability that tests may develop still greater load-carrying ability without excessive settlement or other difficulties.
3. Settlement is essentially proportional to load and takes place as load is applied; it practically ceases thereafter.
4. Sand embankments composed of river-valley and glacial outwash material settle in permanent positions when washed into place with water.
5. This available embankment material is deficient in fines for the usual impervious earth core design, and must be supplemented by some impervious element forming a part of the embankment.
6. Embankments of this material on good sand and gravel foundations can probably be constructed satisfactorily for heads greater than 100 ft by rearranging the foundation cut-off and impervious elements to better structural advantage and with improved economy.
7. Penstock structures properly designed in articulated sections for settlement, may be built safely through embankments and on foundations as herein described.

HARDY DAM ON THE MUSKEGON RIVER, MICHIGAN
Placed in Operation at Full Head on April 30, 1931

A CONTINUOUS THREE-SPAN STEEL TRUSS BRIDGE; CINCINNATI, OHIO

WITH THE PUBLICATION in the April PROCEEDINGS of the paper by Wilson T. Ballard, M. Am. Soc. C.E., on the Chesapeake and Ohio Railway Bridge over the Ohio River at Cincinnati, the Society adds to the list of engineering works that have already been described in TRANSACTIONS. The paper by Mr. Ballard is intended to supplement that by William H. Burr, M. Am. Soc. C.E., entitled "River Spans of the Cincinnati and Covington Elevated Railway Transfer and Bridge Company," which was published in TRANSACTIONS in 1891. For that paper Professor Burr received the Thomas Fitch Rowland Prize.

The new structure is of interest because it is a three-span continuous bridge. It was designed with a fixed bearing at one end, necessitating an unusually heavy anchorage pier. In the construction of the bridge, a river pier made up of an old stone masonry section on a timber caisson was joined with a new concrete section on a concrete caisson. The author describes these features in detail and gives a clear picture of the construction difficulties in each step as well as the methods by which such difficulties were overcome. Work was begun on the substructure in July 1927, and was completed in October 1928. High river stages during the spring and summer of 1928 caused some interruption in the normal program. Field erection was begun in June 1928, and the bridge was placed in service in March 1929.

AN IMPROVED TYPE OF FLOW METER FOR HYDRAULIC TURBINES

A PAPER that promises to be of great and fundamental importance to hydraulic engineers is that by I. A. Winter, Assoc. M. Am. Soc. C.E., which contains the description of a device designed to make it possible to predict the performance of hydraulic turbines before their installation. Mr. Winter describes the fundamental principles involved in determining the turbine discharge by this new method and discusses at some length the results obtained from model tests in the laboratory and on actual plant installation. He describes the theory and methods of installing the piezomoters and gives directions for making instantaneous, continuous, and integrated water measurements. He goes adequately into the subject of scroll-case design as it applies to the proper installation of the differential pressure taps required by his method. The illustrations accompanying the paper are adequate in every way for a clear understanding of all the problems involved.

As a basis for discussion, Mr. Winter concludes that:

1. Laboratory tests on models of scroll cases indicate that the pressure differences on opposite walls of the scroll case can be used to indicate the rate of flow through the turbine.
2. These pressures follow definite laws of motion, and a flow relation can be established involving fundamentally sound principles.
3. The differential pressures in the scroll case are related to the flow through the turbine, the scroll case having the characteristics of a venturi meter. The characteristics of the two are sufficiently similar to enable the use of standard venturi meter instruments in connection with the scroll case.
4. Plant records indicate that differential pressure tap readings in the scroll case will give consistent values over a period of years.
5. The recording of plant operation can be brought to a high degree of efficiency by the use of water-recording devices.
6. To date, experience with scroll-case taps has indicated that they do not offer a means of determining the rate of flow through a turbine unless the scroll case is first calibrated by other water measurements.

News of Local Sections

Buffalo Section

Officers for the Buffalo Section for 1933 have been elected as follows: Nathan H. Sturdy, President; George F. Unger, Vice-President; and Roswell S. Buck, Secretary-Treasurer.

Central Illinois Section

The Central Illinois Section held its regular bi-monthly technical meeting at the Inman Hotel, Urbana, Ill., on the evening of February 21. After an excellent dinner, a short business session resulted in the appointment of several committees to aid in gathering data for questionnaires recently submitted by the Society. Prof. Raymond E. Davis, of the University of California, gave the address of the evening on the subject, "Investigation of Concrete for Boulder Dam." There were 26 members and guests present.

Central Ohio Section

On January 27, a meeting of the Central Ohio Section was held at the Chittenden Hotel, in Columbus, in conjunction with local sections of the American Institute of Electrical Engineers and the American Society of Mechanical Engineers, the Engineers' Club, and local student groups. An illustrated talk on "The Human Seeing Machine" was given by Dr. Luckiesh, Director of the Lighting Research Laboratory of the General Electric Company, Cleveland, Ohio.

There were 26 in attendance at a meeting of the Section held on February 16. After a routine business session, the meeting was addressed by E. H. Deibel, a member of the Ohio State Legislature. The subject of his talk was "Pending Legislation of Interest to Engineers."

Cincinnati Section

A joint meeting of the Cincinnati Section and the Cincinnati Engineers' Club was held on February 16, with approximately 100 in attendance. The proposed "Ohio Engineers' License Law" was discussed at some length. After this, the speaker of the evening, Clifford M. Stegner, Commissioner of Buildings of the City of Cincinnati, was introduced. He gave an interesting address on the subject, "The New Cincinnati Building Code."

Cleveland Section

Consideration of various business matters occupied part of the session of the luncheon meeting held by the Cleveland Section on February 7. An interesting talk on "Rebuilding Our Cities" was then given by Walter R. McCornack, a prominent Cleveland architect. There were 39 members and guests in attendance at the meeting.

Dayton Section

There were 20 in attendance at a meeting of the Dayton Section, held on January 16. The session was devoted to a discussion of the proposed Engineers Registration Bill. The principal points advanced in the discussion were as follows: (1) the Section is in favor of a registration bill; (2) if impossible to put such a bill through the legislature, the matter should be left to the local committee; and (3) a representative, or representatives, from the local committee should be sent to Columbus, at the expense of the Section, to take part in the drafting of this bill.

There were 24 in attendance at the luncheon meeting of the Dayton Section, held at the Engineers' Club on February 20. An illustrated talk on the making of topographic maps and the use of aerial photographs in the making of maps was given by Capt. Bruce C. Hill, of the U.S. Corps of Engineers.

Georgia Section

The election of officers for the Georgia Section for 1933 has resulted as follows: B. M. Hall, Jr., President; George L. Reed and Oliver H. Lang, Vice-Presidents; and L. F. Bellinger, Secretary-Treasurer.

On January 4, the annual supper-dance was held at the Atlanta Athletic Club in Atlanta, with over 60 in attendance. The regular meeting of the Section was held at the same place on January 9. Upon this occasion the chief speaker was W. H. Weir, Chief Engineer of the Board of Health of the State of Georgia, whose subject was "Sewage Disposal Plants of Georgia." There were also five extemporaneous speakers.

Routine business matters occupied the session of the meeting held by the Georgia Section on February 6. The subject of prequalification of contractors was discussed at some length, and a committee of three was appointed to investigate the question of possible legislation requiring bonding companies to determine technical and financial qualifications of contractors, as proposed by the Texas Section.

Indiana Section

On February 16, a meeting of the Indiana Section was held in Indianapolis, in conjunction with the Indiana Engineering Society, the Indiana Society of Architects, and the Associated Building Contractors of Indiana. The speakers at this meeting included Richard V. Murison, a Chicago architect; W. A. Hanley, Chief Engineer of the Eli Lilly Company, Indianapolis; and D. B. Prentice, President of Rose Polytechnic Institute, Terre Haute. At another meeting, held in conjunction with the Purdue University Student Chapter, M. R. Keefe, of Lebanon, Ind., spoke on "Construction of the Persian Railway."

Officers for the Indiana Section for 1933 have been elected as follows: S. C. Hollister, President; H. S. Morse, Vice-President; and W. A. Knapp, Secretary-Treasurer.

Iowa Section

A meeting of the Iowa Section was held in Des Moines on February 8, with 80 members and guests in attendance. The speakers for the occasion were Prof. A. H. Holt, of the State University of Iowa; and Dean Anson Marston, of Iowa State College. Professor Holt spoke on "The Surveyor and His Legal Equipment," while Dean Marston chose for his subject, "Progress in the Solution of the Mississippi Flood Control Problem." As an added attraction, two reels of motion pictures entitled "Construction Work at the Hoover Dam Site," were shown.

Kansas City Section

A dinner and meeting of the Kansas City Section took place in the Hotel President, Kansas City, Mo., on February 17. After the routine business session, an interesting and instructive address was given by E. B. Black, Director of the Society, who spoke on Society Affairs in general, and on the Annual Meeting. This was followed by the report of the Welfare Committee, given by the chairman of the committee, J. F. Brown. The meeting was concluded by an address on the subject, "Requirements and Trends in Modern Air Conditioning," presented by Samuel R. Lewis, consulting engineer of Chicago.

Kansas State Section

At a meeting held in Topeka on February 16, the Kansas State Section elected the following officers for 1933: F. F. Frazier, President; and O. J. Eidmann, Vice-President. There were 42 present at this meeting.

Metropolitan Section

At its regular meeting held in New York, N.Y., on March 15, the Metropolitan Section heard a series of papers covering the engineering features of the Fulton Street-East River rapid transit tunnel in New York City. This, the longest and most expensive shield-driven tunnel on the subway system, was described by J. B. Snow, Division Engineer of the Board of Transportation, who covered the many interesting details of construction. Robert Ridgway, consulting engineer and former chief engineer of the work, then discussed general features of subway tunnels. Other unique characteristics of the Fulton Street line were treated by John H. Quimby, Designing Engineer of the Board of Transportation, from the standpoint of engineering design; and by Miles I. Killmer, Manager of the Mason and Hanger Company, Inc., of New York City, with special reference to plant requirements. The talks were well illustrated by slides, and refreshments were served at their conclusion. The attendance was about 450.

Miami Section

The Miami Section reports the election of officers for 1933 as follows: M. R. Kays, President; and W. T. Wallis, Jr., Secretary.

Milwaukee Section

At a meeting held on February 9, the Milwaukee Section elected officers for 1933 as follows: Emory D. Roberts, President; and F. W. Ullius, Secretary-Treasurer.

Panama Section

A smoker was held by the Panama Section at the Miramar Club, Panama City, on February 7. The guest of honor was Maj.-Gen. William L. Sibert, retired, who is the only living member of the Isthmian Canal Commission. Another important guest was Robert Ridgway, Past-President of the Society. A talk was given by E. D. Stillwell, Superintendent of the Lock Operating Division of the Panama Canal, on the subject of "Maintenance of the Panama Canal Locks."

Philadelphia Section

The annual social meeting of the Philadelphia Section took place on February 25, under the chairmanship of Lyle L. Jenne, who had arranged an entertaining program. After the presentation of some informal musical numbers, dancing and cards were enjoyed. The main attractions, however, were a roulette wheel and other games of chance, the proceeds from which were turned over to the Technical Service Committee for the relief of unemployed engineers.

Portland (Ore.) Section

The members of the Portland (Ore.) Section, at a meeting held on February 15, heard interesting talks by D. C. Henny and J. C. Stevens, respectively Vice-President and Director of the Society. They spoke on the subject of Society business that came up at the Annual Meeting. The feature of the evening was a series of discussions on the subject of technocracy.

San Diego Section

Members of the San Diego Section, at a meeting held at the Churchill Hotel on February 23, heard Rufus Choate, President of the San Diego Harbor Commission, speak on the development of San Diego Harbor. Mr. Choate outlined plans for future development of this harbor, which ranks as one of the ten finest natural harbors in the world.

San Francisco Section

There were 90 members and guests in attendance at the regular meeting of the San Francisco Section, held on December 20. Various business matters were attended to and committee reports read. After this, the speaker of the evening, Francis Donaldson, consulting engineer of New York, was introduced. His subject was "The George Washington Bridge Over the Hudson River at New York, with Incidental Remarks on Recent Eastern Subaqueous Tunnel Construction."

Seattle Section

On January 30, the annual meeting of the Seattle Section was held at the Engineers' Club. Following dinner, George Trayer, Senior Engineer of the Forest Products Laboratory, of the U.S. Forest Service, Madison, Wis., addressed the members on the subject of recent research work in connection with timber products made at the Forest Products Laboratory.

Spokane Section

The regular monthly noon meeting of the Spokane Section was held at the Davenport Hotel on February 10. The feature of the occasion was an account of the Annual Meeting, given by Ivan C. Crawford, Dean of the College of Engineering at the University of Idaho, Moscow, Idaho.

Syracuse Section

On March 6, the Syracuse Section sponsored a lecture by Sherwin F. Kelly, geologist and geophysicist of New York, on the subject, "Exploring Downwards." The following officers have been elected by this Section for 1933: Frank W. Stephens, President; and Earl F. O'Brien, Secretary.

Utah Section

A meeting of the Utah Section was held at the University Club, in Salt Lake City, on January 13. Included in the business session was the election of officers for 1933, which resulted as follows: G. D. D. Kirkpatrick, President; A. B. Purton and Frank M. Allen, Vice-Presidents; and F. H. Richardson, Secretary-Treasurer. An interesting paper on "The Economic Status of the Engineer" was read by R. B. Ketchum, Dean of the School of Engineering of the University of Utah.

Student Chapter News

Cornell University

The Cornell University Student Chapter has enjoyed numerous interesting meetings during the past semester. On November 1, Homer R. Seely, Resident Engineer for the Port of New York Authority on the construction of the central span of the George Washington Bridge, spoke on the construction of this notable structure. His talk was illustrated by several reels of motion pictures. On January 9, the Chapter was privileged to hear a talk on "Tamin' Ole Man River," given by Lt. C. D. Curran, of the Corps of Engineers of the U.S. Army. Motion pictures illustrating levee and revetment construction were shown in connection with his address.

Ohio Northern University

The week of February 20 was set aside by the College of Engineering of Ohio Northern University, as its annual "Engineers' Week." On February 20, under the auspices of the Ohio Northern University Student Chapter, R. H. Randall, president of the Toledo Section of the Society and of R. H. Randall and Company, Geodetic and Topographic Engineers, gave an interesting address on "The Engineer of Tomorrow—His Work in Public and Semi-Public Works After the Depression." There were 109 in attendance.

Rensselaer Polytechnic Institute

At the February 15 meeting of the Rensselaer Polytechnic Institute Student Chapter, an interesting lecture on "City Planning" was given by Harold M. Lewis, of New York City. Mr. Lewis' talk was largely confined to a survey of conditions in the metropolitan area of New York, with recommendations for the future.

Rhode Island State College

On February 16, the Rhode Island State College Student Chapter enjoyed a meeting, at which Dr. L. J. Smith, Superintendent of Health for the City of Warwick, R.I., delivered a very interesting paper on "The Relationship of Public Health and Engineering." In the discussion which ensued, the following subjects were covered: "The Content of Courses in Sanitary Engineering at This College"; "The Relative Merits of the Construction of Water Supply and Sewage Disposal Systems for the Relief of Unemployment"; and "The Employment of Unemployed Engineers to Make a State-Wide Sanitary Survey."

State University of Iowa

Interesting subjects were discussed at meetings held by the State University of Iowa Student Chapter during February. At the meeting on February 8, a lecture on the Cascade Tunnel was presented by R. Schliekelman, of LeMars, Iowa; on February 15, Forest Young, of Marcellus, Mich., spoke on "The Catskill Water Supply of New York City"; and on February 22, "The Holland Tunnel" was discussed by Elmer Nemmers, of LeMars, Iowa. All of these meetings were very well attended.

University of Pittsburgh

A series of meetings has been enjoyed by the University of Pittsburgh Student Chapter during the past semester. Among the subjects discussed were: "Bridge Planning," by V. R. Covell Chief Engineer of the Bureau of Bridges, Department of Public Works, Allegheny County; "Engineering and Traffic Safety," by Lewis W. McIntyre, head of the Department of Traffic Planning, City of Pittsburgh; "City Planning," by C. M. Reppert, Chief Engineer of the Department of Public Works, City of Pittsburgh; and "Tar and Roll Oils" by F. L. Swanberg, of the American Tar Products Company.

Appointments of Society Representatives

Thorndike Saville, C. B. Burdick, and W. L. Stevenson, Members Am. Soc. C.E., together with similar representatives from the American Institute of Chemical Engineers, have been appointed to study the advisability of forming a Joint Committee on the Industrial Pollution of Streams.

George W. Fuller and Otis E. Hovey, Members Am. Soc. C.E., have been reappointed the Society's representatives on the Engineering Foundation.

J. A. Ely, M. Am. Soc. C.E., has been appointed to represent the Society at the commemoration exercises of the Thirty-Seventh Anniversary of Chiao-Tung University and Engineering and Railway Exhibition, to be held from March 30 to April 8, in Shanghai, China.

American Engineering Council

National representative of 26 engineering societies, with a constituent membership of 60,000 professional engineers, reports civil engineering news of the Federal Government

72d Congress Ends

With the adjournment sine die of the 72d Congress, there lapsed an immense volume of legislation, some of which was of vital interest to engineers. The Wagner bill, liberalizing the Emergency Relief and Construction Act of 1932, in accordance with the recommendations expressed by the engineering profession, received favorable action on the part of the Senate Committee but died in the House Committee on Banking and Currency. Senator Wagner has indicated his intention to reintroduce this legislation and vigorously push it at the earliest opportunity.

The Treasury and Post Office Appropriation bill, which finally received the approval of both houses and was signed by the President, continues many of the economy provisions contained in the similar measure passed in connection with 1933 appropriations. Broad powers for reorganization are granted to the President, who may: (*a*) reduce expenditures to the fullest extent consistent with the efficient operation of the Government; (*b*) increase the efficiency of the operations of the Government to the fullest extent practicable within the revenues; (*c*) group, coordinate, and consolidate executive and administrative agencies of the Government as nearly as may be, according to major purposes; (*d*) reduce the number of such agencies by consolidating those having similar functions under a single head, and by abolishing such agencies and/or such functions thereof as may not be necessary for the efficient conduct of the Government; (*e*) eliminate overlapping and duplication of effort; and (*f*) segregate regulatory agencies and functions from those of an administrative and executive character.

Under the power thus vested in him, the President may by executive order: (*a*) transfer the whole or any part of any executive agency and/or the functions thereof to the jurisdiction and control of any other executive agency; (*b*) consolidate the functions vested in any executive agency; (*c*) abolish the whole or any part of any executive agency and/or the functions thereof; and (*d*) designate and fix the name and functions of any consolidated activity or executive agency and the title, powers, and duties of its executive head; except that the President shall not have authority to abolish or transfer an executive department and/or all the functions thereof.

The executive orders must be submitted to Congress while in session and shall not become effective until after the expiration of 60 calendar days after such transmissions, unless Congress shall by law provide for an earlier effective date of such executive order or orders. As it will be necessary for Congress to express its disapproval of any executive orders by legislative means, the President will retain his power to veto such disapproval. The net result will be that the executive orders will become effective unless a two-thirds majority can be mustered in each house in disapproval. The President has delegated the Director of the Budget to study reorganization, and studies are now going forward on definite plans. From these it is expected that executive recommendations will later be made.

Great Lakes-St. Lawrence Deep Waterway Treaty

The International Treaty providing for joint construction by the United States and Canada for a deep waterway from the Great Lakes to the Atlantic Ocean was favorably reported to the Senate on February 22 by its Committee on Foreign Relations. This action followed the adjustment with New York authorities of details concerning the division of costs and reservation of power rights. The report of the committee failed to receive consideration during the last few days of the 72d Congress but it is expected that the matter will be again advanced as soon as practicable during the first session of the new Congress.

Government Competition with Private Enterprise

The Special House Committee to investigate Government competition with private enterprise appointed pursuant to House Resolution 235, presented its report on February 8.

In the course of its hearings testimony was received from representatives of the American Engineering Council, the American Society of Civil Engineers, the American Institute of Architects, the Associated General Contractors of America, and others connected with the engineering profession and construction industry. Among the 29 definite recommendations of the committee, the following are of particular interest to the engineering profession:

The committee recommends that all architectural supervision, including the activities of the Supervising Architect's Office, of the Treasury, the Veterans' Administration, the War, Navy, and other departments, should be centralized and stress placed upon general administration, rather than upon the drawing of plans and local supervision. It recommends that in the drawing of plans and in arranging for supervision of construction in different sections of the country, provision be made for the employment of resident architects of ability. The same principle should be followed by the Supervising Architect's Office, and other departments of the Government, in employing engineering assistance. Wherever practicable, local engineers should be retained.

The committee recommends for reasons of economy that the Corps of Army Engineers, the Reclamation Service, and other Government departments and bureaus engaged in constructive work should, so far as possible, enter into contracts with private competitive bidders and discontinue, as far as possible, the performance of work with Government personnel and Government-purchased equipment. It is the belief of the committee that when actual costs are determined it will be found that a great deal of dredging, harbor work, flood control, and levee work can be awarded to private industry on competitive bids at a substantial saving to the Government.

The committee recommends that the Government departments and agencies be required by Congress to install and maintain a uniform system of accounting, containing all the elements of cost finding that the best practice has sanctioned for public utilities and private industrial and commercial enterprises. The establishment of such governmental accounting methods will also make possible a comparison between public and private costs of production and of service.

The committee recommends that the Federal department engaged in the production of lithographic work and maps, in pricing their products which enter into competition with private business, should include creative and administrative costs of production which private competitors must include in their costs and prices.

Contract for Southern California Water Supply Approved

Approval of a contract between the United States and the Metropolitan Water District of Southern California, involving the construction of Parker Dam on the Colorado River below the mouth of Bill Williams River, was announced by the Department of the Interior on February 1. It is estimated that the dam will cost $13,000,000 and that it will make possible the development of 80,000 hp. The project will not only serve the Metropolitan Water District but will also benefit the Indian and public lands in Arizona. Money for the construction is to be supplied by the Water District and the construction will be carried on by the United States, which will retain ownership of the dam and one-half the power privileges. Following this announcement, Governor Moeur of Arizona informed the Secretary of the Interior that the State of Arizona will oppose by all legal means the proposed construction of dams for the diversion of water from the Colorado River for the Metropolitan Water District of Southern California.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for May

HIGHWAY TRAFFIC between Albany and Rensselaer, N.Y., increased so greatly in density and weight that the bridge over the Hudson River at Albany became entirely inadequate. The State Highway Department found it necessary to replace the old structure with a modern bridge that would at the same time provide for the passage of river traffic. The lift span of 341 ft weighs 3,000 tons—it is the heaviest yet built—and when raised to its maximum height provides an underclearance of 135 ft. The special features of this recent example of a heavy-lift bridge are to be described in the May issue of CIVIL ENGINEERING by Shortridge Hardesty, M. Am. Soc. C.E., whose firm acted in a consulting capacity on the $2,500,000 project.

So many public works planned by engineers depend basically on accurate forecasts of population growth and trends, that the article by Prof. W. R. Tylor, of the Sociological Department of the University of Illinois, also to appear in the May issue, will be helpful in solving many individual engineering problems. He gives an answer to such questions as: Will the present upward sweep of population curves continue in metropolitan areas? Is the movement of people from the rural to the urban districts to continue? Will the decline in the rate of natural increase in cities be more than offset by the migration of rural population to the cities? Is urban population approaching a fixed status?

Across the Madawaska River at the outlet of Lake Temiscouata, in the Province of Quebec, a low sluice weir has been built to store and regulate water for power purposes and to increase the flow to the plant of the St. John River Power Company further downstream. The weir itself is a reinforced concrete articulated structure resting on a gravel foundation, and by means of stop-logs it will raise the water level about 8 ft. In an article planned for the May issue, Jasper H. Ings, Jun. Am. Soc. C.E., one of the designers of the dam, will take up the design of the three parts of this weir—the sluiceway, the apron, and the rolled earth-fill embankment—with special emphasis on the reasons for the design adopted.

At the annual dinner of the past and present officers of the Society, held in New York at the time of the Annual Meeting, Edward P. Lupfer, M. Am. Soc. C.E., traced the development of transcontinental transportation from the opening of the Oregon Trail by those intrepid explorers, Lewis and Clark, to the driving of the last spike in the first transcontinental railroad near Promontory, Utah, in 1869. The May number will include this article in condensed form. Mr. Lupfer calls attention to the train of blooded Arabian camels used to cross the deserts of Texas, New Mexico, and Arizona, by the War Department while Jefferson Davis was Secretary of War; the Pony Express service, which relayed the news of President Lincoln's first inaugural from the Missouri River to the Pacific, 2,000 miles away, in $7^3/_4$ days; and the action of Daniel Webster, who denounced in the U.S. Senate the very idea of a transcontinental railroad.

Pedestrians, it seems from court decisions, have as much right to the use of the highways and streets as motor vehicles, yet the sheer speed and weight of the latter give them a great advantage. Some cities having a controlled traffic system provide for the clearance of pedestrians at intersections by allowing an intermediate period between the red and the green light. In his forthcoming article, A. H. Blanchard, M. Am. Soc. C.E., cites court decisions which have defined the rights of pedestrians and describes the means adopted in various states and communities to decrease the appalling number of accidents, many of them fatal.

Also in the May issue, J. A. Fraps, M. Am. Soc. C.E., will complete his presentation of Arizona's official code for the design and construction of dams in that state. The articles here described, together with others now in preparation, discussions of those previously published, and items of Society interest, will comprise the next issue of CIVIL ENGINEERING.

Early Engineering Books

WHAT PRINTED SOURCES were available to James Geddes, Benjamin Wright, and Charles Brodhead, which might have helped them to solve the many problems that arose during the building of the Erie Canal? To what extent, if at all, could the printed descriptions of French and English bridges have affected the design of such structures in America? In order to answer these and similar questions an exhaustive study would have to be made of the printed sources of information available to the early American engineers. It is of interest to note that such a study has been made and that as a result a bibliography, entitled "Engineering Books Available in America Prior to 1830," is appearing serially in the *Bulletin of the New York Public Library*.

In making this bibliography, the compiler, Ralph R. Shaw, of the Engineering Societies' Library, had to determine, first, the period it should cover. A number of important engineering developments took place just prior to 1830, and these strengthen his claim that 1830 marks the close of the first definite period in American engineering. Among these developments, as listed in the January 1933 issue of the *Bulletin*, were: the first functioning of West Point as an advanced scientific school in 1817; the establishment of the first three artificial gas plants in America between 1816 and 1823; the founding of Franklin Institute in Philadelphia, and of the Rensselaer Polytechnic Institute, at Troy, N.Y., in 1824; completion of the Erie Canal in 1825; granting of the first public railroad charter to the Mohawk and Hudson Railroad in 1826; passage of the first permanent river and harbor bill by Congress in 1826; building of the first railroad bridge in America in 1830.

After the time limit of the bibliography had been determined, there still remained the difficulty of selecting the material, for in that early period there was no sharp dividing line between engineering and other fields. Lacking a final and generally accepted standard, the basis of selection was publications of interest to the engineer in his professional capacity. This called for the inclusion of much that is of secondary importance, technically speaking, but which may have been all the early American engineer could lay his hands on at the time.

Many engineers will be interested in this compilation of early engineering sources, and some may have suggestions to make in the form of additions to the bibliography. Mr. Shaw, who may be addressed at the Engineering Societies Library, 29 West 39th Street, New York, N.Y., will welcome any such suggestions before final publication. The study will be issued later in a separate volume by the New York Public Library.

Copies of the *Bulletin of the New York Public Library*, a monthly publication, may be secured from the library at Fifth Avenue and 42d Street, New York, N.Y., for 10 cents each, or $1.00 per year. Publication of the bibliography was begun in the January 1933 issue.

On the Instalment Plan

ONE OF THE YOUNGER Society members in the Middle West has a unique method of providing for attending Society Meetings. Realizing that the expense of going to the Annual Meeting, for example, especially so soon after Christmas, may be prohibitive, he plans well in advance.

Borrowing a leaf from the book of the popular Christmas funds, he makes a periodic deposit in a separate account, which is intended solely for this use. About two dollars or less per week accumulates during the course of the year to provide a fund which gives him a comfortable, although not an extravagant, trip to New York for the Annual Meeting.

He finds the advantage well repays his efforts and furthermore that the accumulation of the sum required is practically painless. With modification to meet the individual case, his scheme might well be used by many other members.

Code of Standard Practice

Governing Professional Structural Engineering Services

Adopted by the Structural Engineers Association of Southern California
Los Angeles, February, 3, 1932

In California the registration law for engineers provides for civil engineers only; but because the similar registration law for architects places a limitation on practice in the structural engineering field, that is, for "Structural Engineers," there is obviously the necessity in this particular instance for the clarification of such designation. This is provided in the state law by including structural engineers within the definition covering registered civil engineers. This local relation should be kept in mind in order fully to understand the wide applicability of the code here given.

Under date of February 3, 1932, these "Structural Engineers," or at least those who formed an association in southern California, issued a code of standard practice, which is here reproduced in full. The members of the association, operating under this code, state that during the past year it has proved itself very beneficial. They therefore recommend it for adoption to engineers, architects, contractors, and business men throughout the country.

Section I—General

A. *The purpose* of this Code is to provide a standard for the professional relations between a Structural Engineer and his client. It is mandatory in all its details upon all members of signatory bodies unless otherwise specifically so stated in a written contract between the parties concerned.

B. *Definitions* of terms as used in this Code

1. A Structural Engineer is one who by virtue of proper technical education, training, and experience, and by registration as a Civil Engineer in California, is qualified to render professional services as herein defined.

2. A Structural Member is any element of a building, edifice, bridge, tower, frame, or other construction work which transmits stress or carries load other than its own weight.

C. *All structural drawings shall be designed* by the person in responsible charge of the structural design. The signature of a Structural Engineer on any drawing not made by himself or under his supervision shall indicate that the drawing has been completely checked for design and details by the signer.

D. *Structural Engineering Services* are divided into nine classifications as given in Sections II to X, inclusive, and may consist of part or all of the services as defined in those sections.

Section II—Consultation

Consultation shall consist of oral or written discussions with the client regarding any of the following:

1. Types of construction.
2. Methods of construction.
3. Selection and use of materials.
4. Location and arrangement of structural members.
5. Feasible and unfeasible features of architectural design from the structural point of view.
6. Condition of existing structures.
7. Cost of construction.
8. Hazards of the elements.
9. Methods of structural analysis.
10. Subsurface conditions of site.
11. Any other phase of a structural problem on which the client may seek information and which the Structural Engineer is competent to discuss.

Section III—Structural Design

A. *Structural design* shall consist of the determination of the materials, size, shape, strength, and relative position of structural members.

B. *In making the design* the Structural Engineer shall be governed by the recognized and accepted principles of modern sound engineering practice in addition to the limitations and restrictions of a particular building code.

C. *Proper consideration* shall be given to the effect of continuity and fixity of structural members.

D. *Proper consideration* shall be given to the various possible distributions of live loads.

E. *Foundation design* shall be based on the results of a careful study and evaluation of the resistance of the supporting soils on which the structure is to rest.

F. *The safety of the structure* shall be the major consideration in the design. Economy shall be effected wherever feasible within the limits of the structural and architectural designs.

G. *The Structural Engineer* shall keep an adequate and orderly record of the design computations. This record shall identify the designed members and shall show the assumptions as to applied loads and other factors used in the design.

Section IV—Structural Drawings

A. *General*

1. The drawings furnished a client by a Structural Engineer shall illustrate the details of the structure in such a manner that the structural members may be correctly and efficiently installed and erected.

2. The drawings shall be clearly and neatly made to a scale sufficiently large to accurately show the form, location, both horizontally and vertically, the kind of material and the identification marks of all the structural members, together with notes pertaining thereto.

3. It shall be the duty of the Structural Engineer to see that the structural drawings are correlated with those of the Architects and Mechanical Engineers. The location of pipe sleeves, thimbles, hangers, brackets and supports, or of anchors for mechanical equipment and minor architectural trim, need not be shown on the structural drawings, unless their exact location bears an important relation to the structural design.

4. A full schedule of all the working unit stresses employed in the design or a reference to the particular building code governing the design shall appear on the drawings.

5. If designed to resist wind or earthquake forces the assumed intensity of such forces shall be noted on the drawing.

6. In general the structural plans and drawings shall consist of:

a. A foundation plan.
b. A framing plan of each different floor, roof, or framed level.
c. Column details or schedule.
d. Beam details or schedule.
e. Details of penthouse, stair framing, or any other details, schedules, or notes necessary to readily and correctly interpret the drawings.

B. *Foundation Drawings*

In addition to the general conditions of this Section, the foundation drawings shall show:

1. The location of property lines and building lines.

2. The location of the columns and walls at the level of the top of the foundation.

3. The location of each pit, sump, or tunnel below the ground-floor level.

4. Schedules and details showing:
a. Dimensions of the foundations.
b. The sustained load at the top of each foundation.
c. The location and size of reinforcing steel.
d. Details of grillages.
e. Details of sumps, pits, and tunnels.

5. Notes covering:
a. Assumed soil pressure used in the design and the character of the foundation material.
b. Finished grade outside and adjacent to the structure, and inside the structure where paving is not used.
c. Elevation of floors resting on the soil.

C. *Floor Framing Drawings*

In addition to the general conditions of this Section, the floor framing drawings shall show:

1. The location of the property lines and building lines at the level at which the plan is taken.

2. The location in plan and elevation of every structural member at or near the floor level at which the plan is taken.

3. The location of columns and walls at the elevation at which the plan is taken.

4. The location of each framed opening through the floor.

5. The character and thickness of the floor surfaces.

6. The size, weight, and section of all steel beams.

7. The location, size, and method of anchorage of all structural steel masonry supports.

8. Identification marks for all concrete framing members.

9. Details of structural members in the vicinity of the elevation at which the plan is taken, as required to properly illustrate the work.

10. Schedules, details, or notes covering:

a. Typical connections of steel beams.

b. Special connections of steel beams.

c. Rivet and bolt sizes.

d. Concrete slabs, beams, and other concrete structural members, including:

(1) Cross sectional dimensions of the members.

(2) Load, shears or reactions.

(3) Maximum bending moments.

(4) Size, location, length, and bending for reinforcing bars.

(5) Size and location of stirrups.

D. *Column Schedules or Drawings*

1. Column schedules or drawings shall show the total calculated load at each story for each column in the structure.

2. Structural steel column schedules or drawings shall show the size, weight, section and length of each tier of each column. All column splices shall be located and their detail fully determined by drawings, schedules, or notes. The size, detail, and anchorage of each column base or bearing plate shall be shown.

3. Concrete column schedules or drawings shall show the over-all size of the rough column, the core size, the number, size, and length of the vertical bars, the size and spacing of the ties or spiral reinforcement, and the number, size, and length of dowel bars. Special, unusual, or irregular column shapes, and unsymmetrical bar arrangements shall be detailed.

E. *Trusses and Arches*

1. Trusses and arches shall be so detailed that they may be accurately constructed and erected.

2. The stress diagrams for the complete graphical analysis shall be shown and the force scales indicated.

3. The magnitude and sign of the total stress in each member shall be shown on the drawing.

4. Statically indeterminate trusses and arches shall be accompanied by bending-moment diagrams on which the magnitude and sign of the maximum moments shall be shown.

F. *Stairs*

Each different stair shall be detailed. The details shall include:

a. Number of steps.

b. Rise and run of each flight.

c. Stringer and landing beams.

d. Slab thickness and reinforcement for concrete stairs.

e. Treads and connection details for steel stairs.

G. *Walls*

1. All reinforced concrete bearing walls and all other walls which carry load other than their own weight shall be shown in sufficient detail, either by drawings, notes, or schedules, to clearly indicate the material, thickness, height, length, offsets, batter, inclination, reinforcing, and other features important to their stability and necessary for their construction.

2. Non-bearing walls, including partitions and filler walls other than reinforced concrete, need not be shown in detail.

3. Where both architectural and structural plans are prepared for a building, the window and door openings need not be located on the structural drawings unless the walls have been designed as structural members.

4. Lintels over openings in masonry walls may be located by reference to the architectural plans.

SECTION V—STRUCTURAL SPECIFICATIONS

A. *The structural specifications* shall state:

1. The kind and quality of the materials.

2. The testing requirements.

3. The proportions of mixtures.

4. The general methods of fabrication, erection, and installation.

5. The requirements governing the protection of the work in place.

6. The preparation and location of surfaces at which concrete pouring may be temporarily stopped and the provisions for continuing the work.

B. *The Structural Engineer* shall not specify any definite make or brand of material or any patented method of construction without the knowledge and consent of his client or without a provision for the substitution of satisfactory alternates.

SECTION VI—REVIEW AND CHECK OF STRUCTURAL DESIGN, DRAWINGS, AND SPECIFICATIONS

A. *The services* to be rendered under this heading shall vary depending upon the purpose for which it is desired. This work shall be executed without prejudice or bias as to personnel or materials.

B. *The report* on such a check or review shall clearly differentiate between differences of opinion, if such are stated, and errors of calculation. It shall also state the purpose for which the check has been made, whether the check has been general, detailed in part, completely detailed, for safety only, for compliance with a particular building code, for accuracy of dimensions or for economy of construction.

SECTION VII—SUPERVISION

Supervision is defined as the intermittent examination of the construction work at critical periods during the building of a structure and the issuance of instructions governing the conduct of the work.

SECTION VIII—INSPECTION

Inspection of construction work is defined as the complete detailed superintendence of the structural materials and workmanship entering into a structure.

SECTION IX—RESEARCH INVESTIGATION AND REPORTS

Since the services coming under this heading vary in extent and magnitude with the purpose for which they are made, the report shall indicate the degree of thoroughness with which the services have been rendered. Opinions shall be classified as to whether they are based upon judgment, experience, mathematical calculation, or reference to other authority.

SECTION X—LETTING AND DRAFTING OF CONSTRUCTION CONTRACTS

A. *The letting* of construction contracts consists of the preparation of bid forms, the selection of bidders, the analysis of bids received, and the awarding of the contract.

B. *The drafting* of construction contracts consists, in addition to other items, of a detailed statement of the reciprocal relations between the Owner, Architect, and Structural Engineer, the labor to be performed, the materials to be supplied, the time agreed upon for the completion of such work, the responsibility of the Contractor, and the amount and manner in which the Contractor is to be paid for the materials furnished and the labor performed.

C. *The services* of the Structural Engineer shall not include responsibility for the legal phrasing of contracts and bid forms made under his supervision.

Exchange of Hydraulic Research Data

IN ORDER to facilitate the exchange of hydraulic research information in the United States, a clearing house medium has recently been proposed, by which hydraulic laboratories in this country would render quarterly reports of researches under way and planned to the U.S. Bureau of Standards at Washington, D.C. This proposal is the first step in the larger program of the World Power Conference to establish an international exchange of information on hydraulic research.

Under the arrangement proposed, laboratories in the United States would furnish a yearly report on the research facilities at their command. On completion of any research project, a report on it would also be rendered, together with the supporting data and a brief résumé of the results. In exchange for this information, the Bureau of Standards would compile and distribute to the cooperating laboratories a quarterly digest of all the reports received during the preceding quarter and also make the full reports available. Annually a report on the facilities at each laboratory and the type of research it is best equipped to handle would be distributed to all cooperating laboratories.

If arrangements are completed for the establishment of an international exchange of hydraulic laboratory research in-

formation, the Bureau of Standards has agreed to act as the medium for receiving the reports and making them available periodically.

Invitation is extended to public, institutional, and private laboratories to co-operate in the proposed plan to the fullest extent consistent with the conditions under which their work is conducted. Communications regarding the plan should be directed to the U.S. Bureau of Standards, in Washington, D.C., or to O. C. Merrill, M. Am. Soc. C.E., Chairman of the American Committee of the World Power Conference, 405 Lexington Avenue, Room 1419, New York, N.Y.

John R. Freeman Continued Work of Yu

The monthly *Journal of the Association of Chinese and American Engineers* (Peiping, China), for January 1933, is dedicated entirely to the late John R. Freeman, Hon. M. Am. Soc. C.E. In it are tributes to his memory and acknowledgments of the great value of his work in China. These tributes come from both Chinese and American engineers, who were impressed by their contacts with Mr. Freeman. The volume is edited by O. J. Todd, M. Am. Soc. C.E., secretary of the association, whom Mr. Freeman took to China with him in 1919.

The January number of the *Journal* includes a letter by Charles R. Crane, former U.S. Minister to China, written to *The Providence Journal* and reprinted from the October 12, 1932 number of that publication.

In this letter Mr. Crane related some of the events connected with Mr. Freeman's work in China. When, in 1917, the American Red Cross offered Mr. Freeman a commission to make a thorough scientific survey of the Yellow River to learn what could be done to stop its frequent ravages, he hesitated to accept because of the size and importance of the work. When he asked Mr. Crane's advice in the matter, Mr. Crane told him the following legend, as related in the previously mentioned letter:

"Over four thousand years ago there was a vast flood starting way up in the interior of old China. One of the wisest and most esteemed of China's emperors, Shun, after a long search discovered a young engineer by the name of Yu and asked him to try to put the Yellow River within bounds. Yu devoted 16 years to the work, rarely seeing his family, and succeeded in diverting the Yellow River into an alternate channel in time of flood and established a great system of canals which not only could be used in irrigation, but established a wonderful system of transportation, and today, after four thousand years, millions of people still feel the benefit of Yu's work. It is doubtful if anywhere in history one man's beneficent work has touched so many people. The devotion, wisdom, and industry with which Yu served his country made so much of an impression that he was selected to be Shun's successor, and as Emperor Yu became one of China's famous rulers. However, Yu's work was particularly in the back country and since then the Chinese have descended into the lower valleys and that part of the Yellow River is still out of control."

To quote further from Mr. Crane's letter, the outcome was that:

"As Mr. Freeman was impressed with this story and the vast far-reaching influence of a work of peaceful achievement in China, he finally concluded that it was his duty at any cost to go and continue the work of Yu. He took a body of trained young engineers with him and himself devoted long hours and intense interest to this task for three years. He showed how the Yellow River and one or two other wild rivers of China could be controlled and a vast extent of land brought into use through new irrigating canals, and how the enhanced value of this land ought to more than pay the construction costs."

NEWS OF ENGINEERS

From Correspondence and Society Files

William P. Lipscomb, who was formerly in the employ of the Missouri Pacific Lines, has now accepted a position as engineer in the St. Louis County Engineering Department, with headquarters in Clayton, Mo.

Laurence Hussey has resigned his position with the Palos Verdes Estates, at Redondo Beach, Calif., to enter the employ of Olmsted Brothers, of New York, N.Y., on the Fort Tryon Park development.

Charles K. Lewis has accepted a position with the Metropolitan Water District of Beaumont, Calif.

Samuel M. Ellsworth has established a consulting engineering practice at 12 Pearl Street, Boston, Mass. He was formerly Senior Assistant Engineer for Metcalf and Eddy of the same city.

W. McLean Bingley, formerly in the employ of the Dorr Company, has recently been appointed Assistant Sanitary Engineer of the Chlorine Institute, Inc., of New York, N.Y.

J. H. Dowling has resigned his position as Lime Rock Engineer with the Georgia State Highway Department to become Office Engineer of the Florida State Road Department, with headquarters in Tallahassee, Fla.

Leo Charm has accepted an engineering position with the Mill-Rock Construction Corporation, of New York, N.Y. He was previously employed by the Stanley Contracting Company, Inc., of Brooklyn, N.Y.

Ernst L. Pflanz has resigned his position as Principal Assistant Sanitary Engineer of the Sanitary District of Chicago to become Superintendent of Water Works of Glencoe, Ill.

Henry A. Sargent recently accepted a connection with the U.S. Waterways Experiment Station, with headquarters in Vicksburg, Miss. Prior to that he was Office Engineer for the Phoenix Utility Company, of Hot Springs National Park, Ark.

W. H. Statham has severed his connection with the Eastern Vibro Concrete Pile Company of Newark, N.J., to enter the employ of the British Steel Piling Company, Ltd., of London, England.

R. M. Conner has accepted a position with the W. E. Callahan Construction Company and Peterson, Shirley and Gunther, contractors on the Madden Dam Project, Madden Dam, Canal Zone. Prior to that he was Superintendent of Construction for the J. G. White Engineering Corporation, with headquarters in Camaron, N.L., Mexico.

Glen E. Logan has accepted a position with the U.S. Coast and Geodetic Survey, with headquarters in Berkeley, Calif.

Fred W. Morrill, Assistant Engineer with the Ferro Concrete Construction Company, of Cincinnati, Ohio, has taken the class in structural design on bridges at the University of Cincinnati during the sabbatical leave of Prof. Howard B. Luther.

Andrew Weiss, until recently Resident Engineer at Chihuahua, Mexico, for the J. G. White Engineering Corporation, has now entered the employ of the Comision Nacional de Irrigacion, with headquarters in Mexico City.

Don R. S. DeWitt has resigned his position as inspector in the Public Service Department of Long Beach, Calif., to accept a position as draftsman for the Metropolitan Water District of Southern California. He will be located in Beaumont, Calif.

Clarke K. Harvey, who was formerly employed as Principal Assistant Engineer in the Bureau of Bridges of Allegheny County, Pennsylvania, has now been made Bridge Maintenance Engineer for the same county, with headquarters in Pittsburgh.

Frank T. Sheets has resigned his position as Chief Highway Engineer for the Division of Highways, State of Illinois, to set up a consulting engineering practice in Springfield, Ill.

Russell G. Cone, formerly General-Manager of the Tacony-Palmyra Bridge Company, of Palmyra, N.J., has accepted a position as Resident Engineer on the Golden Gate Bridge, with headquarters in San Francisco, Calif.

Henry Cyrus Porter has resigned his position as Resident Engineer with the Texas State Highway Department to become Engineer of Soils and Research in the same department. His headquarters are in Austin, Tex.

Francis H. Wisewell announces the opening of offices as a consulting engineer at 42 East Avenue, Rochester, N.Y. He will specialize in design, construction, operation, and valuation problems of power and industrial plants and in the design of steel structures.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From February 10 to March 9, 1933, Inclusive

Additions to Membership

Andersen, Roy Gibm (Jun. '32), 3326 North Missouri Ave., Portland, Ore.

Campbell, Robert Leland (Jun. '32), 811 East High, Jefferson City, Mo.

Eliasson, Klas Erik Gunnar (Assoc. M. '33), Designer, New York Edison Co., 4 Irving Pl. (Res., 227 East 203d St.), New York, N.Y.

Ferber, Frederick Valentine, Jr. (Jun. '33), 16 Johnson Ave., Hackensack, N.J.

Fish, Franklin Wakefield, Jr. (Jun. '32), 827 East 2d St., Tucson, Ariz.

Geile, Wilfred George (M. '33), Associate Prof. and Head, Dept. of Constr. Eng., North Carolina State Coll. of Agriculture and Eng., State College Station, Raleigh, N.C.

Gobbrino, Frank Eldon (Jun. '32), Care, U.S. Bureau of Public Roads, Yellowstone Park, Wyo.

Haven, Robert Cooper, Jr. (Jun. '32), Fieldman, Michigan Stream Control Comm., Lansing, Mich.

Johnson, Emil Uno (Jun. '32), Vester-Geta, Åland, Finland.

Johnson, LeRoy Francis (Assoc. M. '33), Div. Engr., State Highway Dept., Lebanon, N.H.

Meadows, Clayton (Jun. '32), 4506 McKinney Ave., Houston, Tex.

Norton, William Frederick (Jun. '32), 3040 Mizona Ave., Miami, Ariz.

Parsons, Paul Gates (Jun. '32), 714 East Grand Ave., Alhambra, Calif.

Sherman, Stewart Israel (Assoc. M. '33), Asst. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, Room 2700 Municipal Bldg., New York, N.Y.

Warren, Paul Edward (Jun. '32), "D" Co., 2d Engrs., Fort Logan, Colo.

Wood, Harold Aaron (Jun. '32), Box 485, Gallup, N.Mex.

Membership Transfers

Belousek, Frank Antone (Jun. '28; Assoc. M. '33), Div. Engr., John Monks & Sons, Ulen & Co., Serres, Macedonia, Greece.

Eichler, Philip Henry, Jr. (Jun. '25; Assoc. M. '32), Engr., Goldberger Raabin Co., Inc., 1718 Fulton St., Brooklyn (Res., 450 Harvard St., Hempstead), N.Y.

Johnson, William Scott (Assoc. M. '36; M. '33), Chf. Public Health Engr. and Director, Div. of Public Health Eng. and Sanitation, State Board of Health, Capitol Bldg., Jefferson City, Mo.

Johntz, Albert Frederick (Assoc. M. '17; M. '33), Chf. Engr., Southern Dist. Office, North-Eastern Constr. Co., 812 Nissen Bldg. (Res., 721 Cloverdale Ave.), Winston-Salem, N.C.

Molther, Francis Rattoone (Assoc. M. '30; M. '32), Gen. Architectural and Eng. Partnership with F. J. Morales, Box 48, Ancon, Canal Zone.

Parisi, Francis Generous (Jun. '26; Assoc. M. '32), 79 Oxford St., Harvard Univ., Cambridge, Mass.

Sawyer, William Lincoln (Jun. '28; Assoc. M. '33), Instr., Civ. Eng., Coll. of Eng., Univ. of Florida (Res., 315 Cedar St.), Gainesville, Fla.

Resignations

Campbell, Paul Frederic, Assoc. M., resigned Feb. 13, '33.

Lobb, William Lauck, Assoc. M., resigned March 8, '33.

Meade, Robert Heber, Jun., resigned Feb. 16, '33.

Rice, Harold Gorton, Assoc. M., resigned Feb. 13, '33.

Weller, John Henry, Assoc. M., resigned Feb. 17, '33.

Wood, Galen Arthur, Assoc. M., resigned Feb. 20, '33.

Deaths

Bishop, Lyman Edgar. Elected Jun., May 4, 1909; Assoc. M., Oct. 31, 1911; M., June 16, 1919; died March 6, 1933.

Carty, John Edward. Elected M., May 28, 1912; died Feb. 5, 1933.

Houston, John Jay Lafayette. Elected Assoc. M., Jan. 2, 1912; died March 7, 1933.

Lewis, Sidney Francis. Elected M., May 4, 1881; died Feb. 8, 1933.

Macredie, John Robert Clarke. Elected Assoc. M., Feb. 28, 1911; M., Dec. 31, 1913; died Dec. 5, 1932.

McGovern, Patrick. Elected Affiliate, July 6, 1920; died Feb. 22, 1933.

McGrath, William John. Elected Jun., Oct. 10, 1921; Assoc. M., Oct. 1, 1928; died Feb. 27, 1933.

Marshall, Cyril Ernest Davis. Elected M., Mar. 6, 1907; M., Apr. 17, 1918; died Feb. 16, 1933.

Talbot, Frank Mayhew. Elected Affiliate May 4, 1909; died March 6, 1933.

Total Membership as of March 9, 1933

Members	5,809
Associate Members	6,312
Corporate Members	12,121
Honorary Members	19
Juniors	2,954
Affiliates	113
Fellows	5
Total	15,212

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 97 of the 1933 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

Construction

Field Engineer; Jun. Am. Soc. C.E.; 28; married; 2½ years university training in civil engineering; 10 years experience in construction of hydro-electric and steam power plants, dams, bridges, and highways. Good fieldman for contractor. Location immaterial. Available immediately. D-1962.

Design

Structural Engineer; Assoc. M. Am. Soc. C.E.; 34; married; 8 years experience in design and construction of pulp and paper mills and in improving and remodeling them. Best references. Open for any professional work in office, field, or mill operation. Location anywhere. Speaks Russian. Naturalized. D-119.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 37; married; graduate; state licenses; 10 years experience in surveying, design, and construction. Railroad, highway, parkway, harbor, and tall building projects. Completely familiar with the design of rigid-frame bridges, reinforced concrete or steel, single or multiple-span, square or skew. Available immediately. D-1496.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 29; married; B.S. and C.E. degrees; licensed professional engineer; 7 years, in February 1933, with one of the largest general contractors in the East; varied office and field experience; capable structural designer, checker, and detailer. Exceptional knowledge of reinforced concrete design and construction. Excellent references. C-894.

Civil Engineer; Jun. Am. Soc. C.E.; 26; B.S. in C.E., 1927; 4½ years experience in railroad structures, hydro-electric developments, and industrial buildings; 6 months experience, clerical work in mill office. Salary is, at present, a secondary consideration. Available immediately. Location immaterial. D-372.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 32; married; New Jersey structural license; B.S. in C.E.; 12½ years structural experience, designing and detailing concrete and steel railroad and highway bridges; flat-slab, steel and timber industrial buildings, and reinforced concrete dams. Desires field or office position. Location immaterial. D-1976.

Civil Engineer; Assoc. M. Am. Soc. C.E.; 34; single; college graduate; licensed in Pennsylvania and New Jersey; 10 years experience in the design and construction of highways, bridges, water works, and municipal structures. Responsible charge of engineering works from inception to completion. Location immaterial (foreign or domestic). Available on short notice. D-1991.

Executive

Civil Engineer; Assoc. M. Am. Soc. C.E.; 30 years experience, chiefly specializing in food markets, cold storage, refrigeration; has had considerable experience as general manager for sales division of electric refrigerators, domestic and commercial. D-1516.

Civil Engineer; M. Am. Soc. C.E.; 51; married; licensed-professional engineer, Indiana; 9 years experience as instrumentman, draftsman, and assistant engineer on railroad maintenance and construction; 11 years experience on highway work as district engineer in charge of general maintenance and of bituminous-type road construction. Location Middle West or West. D-1982.

CONSTRUCTION EXECUTIVE; M. Am. Soc. C.E.; 53; 33 years diversified experience on large construction projects. Steam, hydro-electric stations, dams, oil pipe lines, office buildings, transmission lines, substations, railroads, gas, and water lines. Well equipped for all classes of public utility work. Chief construction executive for past 15 years with three large engineering-contracting organizations. Available immediately. College graduate. D-1995.

EXECUTIVE ENGINEER, MANAGER; M. Am. Soc. C.E.; 37; married; graduate; 15 years on hydro-electric projects and large work. Experienced on all types of foundations, dams, and other structures, domestic and foreign. Qualified to plan and organize plant, and manage for speed and cost. Also estimates for price or other work, preliminary studies, estimates, designs. C-9694.

ENGINEER AND ARCHITECT; M. Am. Soc. C.E.; graduate; licensed. Design and construction of housing groups, commercial garages, factories, chemical plants, power plants. Organization and supervision of office staffs and field forces. Specifications for materials and equipment, and purchase of all materials. B-2835.

CIVIL ENGINEER; M. Am. Soc. C.E.; 15 years general engineering experience in executive and responsible charge of public works construction; 15 years experience in all phases of highway engineering-reconnaissance, location, design, supervision, estimating. Available immediately in any capacity. Salary secondary. B-5666.

ENGINEERING EXECUTIVE; M. Am. Soc. C.E.; Cornell graduate, C.E.; 20 years experience in structural steel and general building construction. With large engineering and construction firm for past 15 years as chief engineer, sales manager, and director. Exceptional qualifications. Interview desired. D-1947.

ENGINEERING EXECUTIVE; Assoc. M. Am. Soc. C.E.; 33; married; 10 years experience in designing, estimating, and supervising construction of buildings. Competent to assume responsibility from start to finish and desires position where this ability and initiative are essential. Available on short notice. Southern California only. D-1970.

REINFORCED CONCRETE ENGINEER AND CONCRETE TECHNICIAN; Jun. Am. Soc. C.E.; 32; married; B.S. in C.E.; 10 years experience in design and construction of pavements, bridges and buildings, concrete control, sales promotion, and office management. Desires position with architect, engineer, contractor, or as cement manufacturers' field engineer. Available immediately. D-1926.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; licensed professional engineer, New York State; 25 years experience in design and construction of steam and hydro-electric power plants, transmission lines of all capacities, including 220,000 volts, industrial plants, electric railways, valuations, estimates, specifications, and purchasing. Desires responsible charge of work. Location New York. B-5423.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 8 years varied structural experience; drafting, design, and construction; also office and selling experience. Capacity to assume responsibility. Desires connection in any capacity, in field or office; preferably with construction company, consultant, architect, or contractor; also instructorship or sales-engineering. C-2605.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; married; licensed New York and Florida; 10 years experience on highways, bridges, parkways, and park design and construction; control and topographic surveys; building design and construction; contractor's superintendent on grading, sewers, water lines, and concrete paving. Available April 1. Location immaterial. C-7024.

ENGINEER; M. Am. Soc. C.E.; 41; industrial, mechanical, and civil, with business training. Experienced in office and field on investigations, reports, plant operation, planning, design, construction, selling, and general business. Experience covers many industries as well as power. Satisfactory references. A-1185.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 37; married; graduate of Columbia University, degree of C.E.; state license; 15 years experience in reinforced concrete, including estimating, designing, and supervision of construction. Thoroughly familiar with all types of reinforced concrete floor systems used in building construction. B-4442.

ENGINEER; Assoc. M. Am. Soc. C.E.; 38; married; degree. Well adapted to development, research, or investigation work in either the technical or sales fields, where extensive experience and broad vision may be utilized, or in cases where the work does not conform with specialized experience and therefore has to be learned. High standards of accomplishment. B-5254.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 37; graduate; over 16 years experience. Past 10 years on pulp and paper mill design, and construction and equipment installation. Previous experience in general building construction, sewers, estimating, and surveying. Available now. B-2343.

STRUCTURAL ENGINEER; M. Am. Soc. C.E.; 45; married. Extensive experience in the handling of estimating, designing, and sales of modern steel fabricating business. Offers exceptional qualifications. C-5095.

STRUCTURAL-HYDRAULIC ENGINEER; Assoc. M. Am. Soc. C.E.; 40; graduate of University of Illinois; registered structural engineer in Illinois; 17 years engineering experience on design of hydraulic structures, highway bridges, mill buildings, and highways; hydraulic design, investigations, and project estimates; construction of bridges and highways. Location preferred Middle West, West, or Southwest. C-9167-3049 Chicago.

JUNIOR

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; B.S. in C.E., Rutgers University, 1930; 1½ years as transitman in Essex County Highway Department; passed New Jersey State Civil Service senior draftsman examination. Desires working or teaching position in any branch of civil engineering, preferably one involving mathematics. D-663.

JUNIOR CIVIL ENGINEER; Jun. Am. Soc. C.E.; 27; single; B.S. in C.E. (structural option) University Illinois, 1931; approximately 2 years experience in drafting and surveying; passed U.S. Civil Service examinations for junior engineer, civil and structural engineering draftsman. Desires position in any branch of civil engineering, preferably structural. Location and salary immaterial. Available immediately. D-1959.

GRADUATE OF THE VIRGINIA POLYTECHNIC INSTITUTE; Jun. Am. Soc. C.E.; 2½ years experience on highway location and construction. Now holding research fellowship at Virginia Polytechnic Institute and doing graduate work. Expects to be awarded M.S. in C.E., in June. Available June 8, 1933. Desires work anywhere in the United States. D-1967.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 24; single; 1931 graduate; speaks Spanish and French; 1 year with New York State Highway Department in surveying, inspection, and estimating. Excellent topographic and structural draftsman. Desires any engineering position. Salary secondary. Location immaterial. D-1979.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 25; B.S. in C.E., 1929. Desires any type of engineering, teaching, or construction position. Has had 2½ years experience as private tutor; as junior metallurgist, and special mill-building layout man in steel mill. D-1911.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; single; B.S. in C.E., Clarkson College of Technology; 2 years experience with the New York State Highway Department on construction of roads and bridges; desires any engineering position. Location anywhere. D-1990.

MISCELLANEOUS

ENGINEER-ACCOUNTANT; Jun. Am. Soc. C.E.; 29; licensed, New York State; graduate training in business administration; 7½ years experience in design and as contact man with field office on New York subways, statistical studies, valuation, estimating. Desires position as all-round man for contractor or appraisers. D-1086.

HYDRAULIC ENGINEER; Assoc. M. Am. Soc. C.E.; 35; married; 13 years experience in investigations, design, and construction of hydro-electric power plants. Capable of supervising the design of all structures and the preparation of all drawings common to hydro-electric developments. Thoroughly familiar with stream regulation for system operation. C-9380.

SALES

STRUCTURAL DESIGNER AND SALES ENGINEER; M. Am. Soc. C.E.; 39; married; graduate C.E.; licensed; 18 years experience in structural designing, managing steel fabricating plant, and selling. Last 8 years head of consulting engineering office, specializing in structural designing for architects, owners, and contractors. Large acquaintance among architects and contractors in northern New Jersey and metropolitan New York. A-5489.

STRUCTURAL AND SALES ENGINEER; Assoc. M. Am. Soc. C.E.; 39; married; graduate. High-class sales executive. Long experience in sales, sales management, designing, detailing, and estimating bridges and buildings. Seeking connection with building material manufacturer or distributor, also engineering position with general contractor. Available immediately. Capable and aggressive. D-16.

TEACHING

ASSOCIATE PROFESSOR OF STRUCTURAL ENGINEERING; Assoc. M. Am. Soc. C. E.; married; degrees B.S., M.S., and C.E., University of Illinois; 3 years experience, teaching major courses, structural theory and design in ranking university; 5 years practice on reinforced-concrete and steel structures—chiefly with Waddell and Hardesty, consultants—cantilever, arch, suspension, movable, continuous-girder spans; technical writer; world traveler. Desires position in leading university. D-330.

RECENT BOOKS

New books of interest to Civil Engineers, recently donated by the publishers to the Engineering Societies Library, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 87 of the Year Book for 1933. The statements made regarding the books are taken from the books themselves, and this Society is not responsible for them.

GEOMETRY OF ENGINEERING DRAWING. By G. J. Hood. 2 ed. New York and London, McGraw-Hill Book Co., 1933. 348 pp., diagrs., charts, 9 × 6 in., cloth, $2.50.

Many changes and additions have been made in this textbook, although the basic method and plan are retained. Almost one thousand new problems have been provided and new material has been introduced. Practical applications are illustrated.

PROBLEMS IN PUBLIC UTILITY ECONOMICS AND MANAGEMENT. By C. O. Ruggles. New York and London, McGraw-Hill Book Co., 1933. 737 pp., charts, tables, 9 × 6 in., cloth, $6.

The case method is here applied to the analysis and solution of problems dealing with economic and business aspects of public utilities. One hundred and twenty problems, based on actual cases, are discussed. These cover problems of production, management, wholesale and retail marketing, valuation, rate making, regulation, etc.

STRUCTURAL ENGINEERING. By J. B. Kirkham. 2 ed. New York and London, McGraw-Hill Book Co., 1933. 759 pp., diagrs., charts, tables, 9 × 6 in., cloth, $5.

This volume is intended as a self-explanatory manual for practical men and for college students. The principles and practice of the design of simple structures are set forth simply, clearly, and fully, without the use of higher mathematics.

CURRENT PERIODICAL LITERATURE

Abstracts of Articles on Civil Engineering Subjects from Magazines in This Country and in Foreign Lands

Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the Library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own files, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, plus postage, or technical translations of the complete text may be obtained at cost.

BRIDGES

AUSTRALIA. Clarence River Bridge—II, J. W. Roberts. *Inst. Engrs. Australia—Journal*, vol. 4, no. 12, Dec. 1932, pp. 405–414. Construction; tenders and contracts; plant contracts; field work; concrete; road approaches; painting; warning devices; lighting; dolphins; signalling. (Concluded.)

BRIDGE PIERS, CONSTRUCTION. Sheeting Interlocked on Barge for Cofferdam Wall. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, p. 191. Construction of single-wall cofferdam of 20-ft steel sheetpiles, for piers for the new multiple-arch concrete highway bridge across the Mohawk River, between Cohoes and Watersford, N.Y.; lack of vertical clearance necessitated the assembling and interlocking of the sheeting from the side of a barge.

BRITISH COLUMBIA. Burrard Bridge, Vancouver, J. R. Grant. *Engineer*, vol. 155, no. 4020, Jan. 27, 1933, pp. 84–85, and 94. High-level highway bridge, with a length of 2,817 ft 6 in. between abutments; central portion consists of four deck-truss spans and through-truss channel span. Between abutments and steel portion of structure, the bridge consists of reinforced concrete viaducts, 353 ft long on the north shore, and 1,294 ft on the south shore.

CALIFORNIA. Timber Falsework 260 Ft High Carries Concrete Highway Bridge During Construction. *Construction Methods*, vol. 15, no. 1, Jan. 1933, p. 17. Main features of method of construction of a reinforced-concrete highway bridge, 740 ft long, comprising a concrete arch 330 ft in span, rising 260 ft above the creek; the bridge crosses Bixby Creek on the Carmel-San Simeon highway, California.

CONCRETE, GREAT BRITAIN. Reinforced Concrete Bridges at Peterborough. *Concrete and Constr. Eng.*, vol. 28, no. 1, Jan. 1933, pp. 27–40. Design and construction of a bridge over the river Nene, consisting of three arch spans, 34 ft, 65 ft, and 46 ft in clear, carried on mass concrete abutments and piers, and of a girder bridge over the L.N.E. Railroad, consisting of two spans, each 37 ft 4 in. in clear, supported on a central pier; retaining walls; form work; quay wall; quantities of materials; cost £100,000.

CONCRETE GIRDER, EXPANSION ROLLERS. Lenticular Expansion Rollers for Concrete Spans, A. B. Willett. *Eng. News-Rec.*, vol. 110, no. 8, Feb. 23, 1933, p. 255. Substituting a lenticular-shaped roller for a circular zone-shaped roller; formulas of action of the roller upon the movement of the girder due to expansion or contraction.

CONCRETE SLAB. Plain Slab Bridge, G. P. Manning. *Concrete and Constr. Eng.*, vol. 28, no. 1, Jan. 1933, pp. 9–11. Features of reinforced concrete slab bridge, trapezoidal in plan, span about 55 ft, 35 ft wide, constructed for Associated London Flour Millers, Ltd., at their Deptford Mills.

GREAT BRITAIN. East Lothian By-Pass. *Surveyor*, vol. 83, no. 2139, Jan. 20, 1933, pp. 51 and 52. Features of reinforced-concrete arch highway bridge, at Dunglass, near Cockburnspath, East Lothian, spanning gorge in a single arch of 162-ft span.

HIGHWAY, LONDON. Bridges of London—Past, Present, and Future, J. Benskin. *Roy. Soc. Arts—Journal*, vol. 81, no. 4186, Feb. 10, 1933, pp. 280–296 (discussion) 296–301. Bridge approaches; national and financial control; architectural and historical preservation; technical considerations; outstanding legislative and administrative events bearing upon construction of bridges across the Thames in London.

PLATE GIRDER, GEORGIA. Rebuilding Fifth St. Bridge at Augusta, Ga., S. B. Slack. *Eng. News-Rec.*, vol. 110, no. 7, Feb. 16, 1933, pp. 214 and 215. Construction of new steel plate-girder bridge over the Savannah River, including 9 cantilever spans, from 72.5 to 120.3 ft in length.

RAILROAD CROSSINGS, Newark, N.J. Three-Level Rail and Road Crossing, A. B. Cohen. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, pp. 179–183. Description of railroad crossing in Newark, N.J., featuring compact combination of structures, including flat-slab railroad bridge of unique design with slab, 32 in. thick, in panels 28 by 35 ft, main state highway on lowest level, and park drive carried overhead on concrete rib arch having skeleton abutments.

REINFORCED CONCRETE. Survey, Location, and Site Investigation for Reinforced Concrete Bridges, A. W. Legat. *Concrete and Constr. Eng.*, vol. 28, no. 2, Feb. 1933, pp. 127–144. Borings and trial holes; provision for mains; flood level and river flow; accommodation roads; utilization of available materials and labor.

STEEL TRUSS, MOVING. Two Three-Span Bridges Moved in Six Hours, L. Brannon. *Roads and Streets*, vol. 76, no. 2, Feb. 1933, pp. 85 and 86. Method of moving steel-truss highway bridges over the Miami River near Dayton, Ohio; traffic stopped only 18 hours.

SUSPENSION, SAN FRANCISCO—OAKLAND. San Francisco-Oakland Bay Bridge. *West. Construction News and Highways Bldr.*, vol. 8, no. 1, Jan. 10, 1933, pp. 2–4. List of 17 major contracts scheduled for early letting on the $78,000,000 San Francisco-Oakland Bay Bridge; description of bridge; total length of bridge project is 43,500 ft; bridge traffic; terminal locations; construction quantities.

VIADUCTS, FLOORS. Viaduct Roadway. *Construction Methods*, vol. 15, no. 1, Jan. 1933, pp. 32–34. Reinforced concrete deck on high-level viaduct across the meadows between Newark and Jersey City; viaduct design; reinforcement erection; welding operations; concrete forms; concreting procedure; mixing and hauling; finishing and curing slab; asphalt block pavement.

BUILDINGS

GREAT BRITAIN. Sheffield City Hall, W. L. Scott. *Concrete and Constr. Eng.*, vol. 28, no. 1, Jan. 1933, pp. 3–8. Design and construction of a 4-story reinforced concrete building, having a total capacity of 2,500,000 cu ft and incorporating five halls; the main hall is elliptical in plan, with a major axis of 135 ft and a minor axis of 102 ft 6 in.; it has a domed ceiling, 60 ft above the ground-floor level, and a seating capacity of 4,000.

POST OFFICES, RAILROAD TERMINALS. Operate Passenger Terminal Under Mammoth Building Project. *Ry. Age*, vol. 94, no. 3, Jan. 21, 1933, pp. 71–74. Construction of 12-story post office over 15 tracks and over platforms of Chicago Union Station; ground dimensions, 796 ft by 344 ft; construction interfered with interlocking; new wiring required; use of train shed as construction platform; steel erection.

WIND BRACING. Wind-Stress Analysis and Moment Distribution, R. Fleming. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, pp. 194 and 195. Application of Prof. Hardy Cross's method of distributing fixed-end moments in analyzing indeterminate frames, to wind problem in tall-building design.

CONCRETE

COLUMNS, DESIGN. Reinforced Concrete Column Investigation. *Am. Concrete Inst.—Journal*, vol. 4, no. 6, Feb. 1933, pp. 283 and 284. Minority recommendation of Committee 105 for design formula of reinforced concrete columns.

CONCRETE LINING FOR TUNNELS. Asphalt-Impregnated Slabs of Concrete in Salt Water, W. R. Shettel. *Eng. News-Rec.*, vol. 110, no. 4, Jan. 26, 1933, pp. 108–109. Use of slabs of precast concrete to line outlet tunnels of the new extension of the Seal Beach steam plant of the Los Angeles Gas and Electric Corp. to prevent the disintegration of concrete by sea water; method of laying up lining slabs against skeleton forms; ceiling lining was laid on usual type of timber forms.

CONSTRUCTION, FORMS. Sliding Form Work. *Am. Concrete Inst.—Journal*, vol. 4, no. 6, Feb. 1933, pp. 285–299. Supplement to the report of Committee 608 on the use of sliding forms in concrete construction; double or hollow wall forms; finishing; winter work; plant of Federal Cold Storage Company of Clayton, Del.; plants of City Ice and Fuel Company at Niles, Mich., Cleveland, Ohio, Pittsburgh, Pa., and Nashville, Tenn.

CREEP. Creep of Concrete Under Load, W. H. Glanville and F. G. Thomas. *Structural Engr.*, vol. 11, (New Series), no. 2, Feb. 1933, pp. 54–69. Properties of plain concrete at ordinary working stresses; results of research at Building Research Station; creep of concrete in compression and tension; effect of cement content; true and "effective" moduli of elasticity; column tests; stresses developed in steel by creep in concrete; movements of cement mortar beams under load; formation of cracks; creep of concrete at high stresses; modular ratios.

DISINTEGRATION. Effect of Mixing Portland and Aluminous Cements, K. E. Dorsch. *Concrete and Constr. Eng.*, vol. 28, no. 2, Feb. 1933, pp. 151–154. Report on tests, made at the Karlsruhe Institute of Technology, on the effect of adding aluminous cement as a protective coating to prevent disintegration of portland cement concrete in salt solutions, etc.

FERRY SLIPS, CONCRETE. Concrete Ferry Landing at North Vancouver, B.C., G. S. Hanes. *Can. Engr.*, vol. 64, no. 4, Jan. 24, 1933, pp. 13 and 14. Description of structure, 22 years old, which has shown no signs of deterioration in salt water.

FRAMED STRUCTURES, CONCRETE. Economics of Concrete Frames, W. S. Wilson. *Civ. Eng. (Lond.)*, vol. 28, no. 317, Jan. 1933, pp. 34, 37, and 38. Study of materials and costs of reinforced concrete frames.

FORMS. Moving Forms for Concrete Construction, H. H. Broughton. *Engineer*, vol. 155, no. 4022, Feb. 10, 1933, pp. 134 and 135. Although the examples described are restricted to grain elevators, this method has been applied with success to storage bins of all kinds, quay walls, etc.; the elementary principle is that of pouring concrete into a shallow mold of the same size in plan as the building and as concrete takes its initial set, slowly and continuously moving mold vertically upwards till the required elevation is reached.

HYDRO-ELECTRIC POWER PLANTS, CONSTRUCTION. Concreting Problems—Chats Falls Power Development, H. L. Trotter and W. Schnarr. *Am. Concrete Inst.—Journal*, vol. 4, no. 6, Feb. 1933, pp. 249–274. Mixing and placing of concrete in the construction of the Ottawa River hydro-electric development, consisting of a concrete dam and sluices 15,000 ft long, averaging 20 ft in height, and a power-house, 500 ft by 72 ft and 133 ft high; system of control; placing large blocks in one continuous operation to heights of 60 ft without horizontal construction joints; winter concreting.

MIXING. Over-a-Minute Mixing Adds No Strength, F. S. Besson. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, pp. 183–185. Cylinder tests of field concrete made by the Nashville Engineer District show no material gain in strength by increasing the mixing time to 2 minutes; results of previous tests.

Curing of Portland Cement Concrete—V, J. M. Portugals. *Can. Engr.*, vol. 64, no. 4, Jan. 24, 1933, pp. 9–12. Effect on strength; production of watertight concrete; effect of age on concrete; interrupted curing; influence of temperature curing; curing process; protection during cold weather; high early-strength concrete.

PUMPING. Record Cement-Pumping Lift Used at Pine Canyon Dam. *Eng. News-Rec.*, vol. 110, no. 8, Feb. 25, 1933, p. 241. Practice in the construction of the concrete dam near Pasadena, Calif; bulk cement, trucked to site, is pumped up to mixing plant 302 ft above.

READY-MIXED CONCRETE PLANTS, MISSOURI. Contractors Enter Ready-Mixed Concrete Field in St. Louis. *Rock Products*, vol. 36, no. 1, Jan. 25, 1933, pp. 48 and 49. Methods and equipment at two ready-mix plants operated by the Concrete Sales Corporation and Contractors Material Company.

SAND AND GRAVEL PLANTS, MASSACHUSETTS. New Sand and Gravel Plant in Boston District, W. R. Carmichael. *Rock Products*, vol. 36, no. 1, Jan. 25, 1933, pp. 19–21. Design, methods, and equipment of the plant of the Concrete Materials Company, at Saugus, Mass.

SETTING. Distinction entre la prise et le durcissement des ciments et des mortiers, E. Marcotte. *Genie Civil*, vol. 102, no. 4, Jan. 28, 1933, pp. 86 and 87. Methods of measuring the speed of setting of cements and cement mortars; resistance to crushing as function of logarithm of time; distinction between setting and hardening of cement.

TESTING, BENDING. Effect of Size of Specimen, Size of Aggregate, and Method of Loading upon Uniformity of Flexural Strength Tests, W. F. Kellermann. *Pub. Roads*, vol. 13, no. 11, Jan. 1933, pp. 177–184. Report on tests made by the U.S. Bureau of Public Roads, the primary purpose of which was to determine which combination of several variations in test procedure gave the most satisfactory results from the standpoint of uniformity; recommendations.

CONSTRUCTION INDUSTRY

COSTS. Unit Bid Summary. *West. Construction News and Highways Bldr.*, vol. 8, no. 2, Jan. 25, 1933, pp. 60, 62, 64, and 66. Unit costs bid on bridges and culverts, water supply systems, and street and road work in Oregon, California, Nevada, and Montana.

UNITED STATES. Materials' Producers Talk of Markets and Technology. *Eng. News-Rec.*, vol. 110, no. 4, Jan. 26, 1933, pp. 120–123. Abstracts of discussion by representatives of paving brick, sand and gravel, crushed stone, and ready-mixed concrete industries at the highway and construction congress held in Detroit.

WESTERN STATES, COSTS. Unit Bid Summary. *West Construction News and Highways Bldr.*, vol. 8, no. 1, Jan. 10, 1933, pp. 28, 30, 32 and 33. Unit bids on street and road work, river and harbor work, and bridges and culverts in California, Oregon, and other Western states.

DAMS

CONCRETE GRAVITY, INDIA. Metur Dam, Madras. *Engineering*, vol. 135, no. 3496, Jan. 13, 1933, p. 55. Brief details of this dam, which will be the largest in the world; length 5,300 ft; height, above average bed level of river, 176 ft; maximum height, above lowest point of foundations, 230 ft; view reproduced shows dam at relatively advanced stage; dam considerably extended as a result of impounded water; other features prominent in view are two connecting towers and new bridge.

CONSTRUCTION. Madden Dam Aggregate Handled by Mile-Long Aerial Tramway, A. J. Ackerman. *Construction Methods*, vol. 15, no. 2, Feb. 1933, pp. 30 and 31. Description of aerial tramway capable of delivering 225 tons of gravel per hour to screening plant for the construction of a concrete dam, 900 ft long and 220 ft in maximum height; the gravel is loaded by Bucyrus-Monighan $2^1/_2$-yd walking dragline into side-dump cars with capacity of 5 cu yd, operating on a narrow-gage track.

EARTH, TESTING. Percolation Slope in Dams Measured by New Device, L. L. Meyer. *Eng. News-Rec.*, vol. 110, no. 4, Jan. 26, 1933, pp. 109 and 110. Device for measuring water level installed by the Los Angeles Water Department in reconstructed Chatsworth Dam No. 2, to determine the percolation gradient in any part of the rolled-earth fill; lines of tubing from well points located in the structure to the recording box on the dam crest where water levels are determined by manometer readings.

ECONOMICS. Economics of Service Reservoir Construction, S. McConnel. *Water and Water Eng.*, vol. 35, no. 413, Jan. 20, 1933, pp. 16–21. General principles of construction; economical proportions of structures, including concrete reservoirs; economy of earth backing.

HYDRAULIC-FILL, COBBLE MOUNTAIN, MASS. Construction of Cobble Mountain Dam of Springfield Water Works, H. H. Hatch. *New England Water Works Ass'n—Journal*, vol. 46, no. 4, Dec. 1932, pp. 327–344. Construction of hydraulic-fill dam, 220 ft high; paper similar in content to several articles indexed in Engineering Index for 1932 from various sources.

MOVABLE. Note sur un type simple de barrage et de prise d'eau pour dérivation au fil de l'eau, R. F. Berthe. *Revue Générale de l'Electricite*, vol. 32, no. 26, Dec. 31, 1932, pp. 873–879. Design and construction of the Bastan diversion weir of the Haute-Pyrénées development of the Société Hydroélectrique de la Cere, consisting of a cylindrical gate 10 m long, for raising water to a height of 3 m above sill; theoretical data on undershot flow from such gates; design of diversion canal.

RESERVOIRS. Quality of Impounded Water Supplies, H. P. Eddy. *New England Water Works Ass'n—Journal*, vol. 46, no. 4, Dec. 1932, pp. 396–405. Location of reservoirs; preparation of reservoir sites; watershed control; effect of storage on quality; reservoir operation; water purification; laboratory supervision.

WEIRS, MOVABLE. Bear-Trap and Hydraulic-Roof Weirs, P. Baumann. *West. Construction News and Highways Bldr.*, vol. 8, no. 2, Jan. 25, 1933, pp. 41–43. Comparative statical analysis of bear-trap and roof weirs of Swiss type.

FLOOD CONTROL

MEXICO. Estudio Preliminar del Control de Avenidas de una Corriente Torrencial, mediante un Vaso Regulador, L. Arteaga. *Irrigacion en Mexico*, vol. 5, no. 8, Dec. 1932, pp. 680–695. Preliminary study of flood control of torrential stream, by means of regulating reservoir, on the Rio Grande in Morelia Valley, state of Michoacan, Mexico.

RIVERS, IMPROVEMENT. Modification of Bell's Bund System Simplifies River Control, P. Claxton. *Indian Eng.*, vol. 93, no. 1, Jan. 7, 1933, pp. 13–15. Outline of the method of river control known as Bell's Bund System, widely used in India for the protection of railroad embankments at river crossings; using still-water pockets to keep swift waters of main stream from reaching banks needing protection; examples.

FOUNDATIONS

BRIDGE PIERS. Bridge Pier Caissons Lowered from Floating Gantry Trusses. *Construction Methods*, vol. 15, no. 2, Feb. 1933, pp. 26–28. Method of positioning and sinking caissons of three river piers on the Albany-Rensselaer highway bridge across the Hudson River; building up and lowering caissons on threaded rods supported from floating gantry trusses.

DESIGN. Advances in Foundation Design, H. F. Blood. *West. Construction News and Highways Bldr.*, vol. 8, no. 3, Feb. 10, 1933, pp. 87–90. Supporting materials; settlement in compressible materials; usual methods for determining safe bearing value; errors in using these methods; spacing of piles; soil investigation previous to starting design; soil properties that engineers should know; sources of information on soil research.

FOOTINGS. Beitrag zur Berechnung von Grundwerksockeln, H. Schuette. *Bautechnik*, vol. 10, no. 52, Dec. 2, 1932, pp. 677 and 678. Theoretical mathematical analysis of the design of stresses in masonry footings; modes of failure of concrete footings.

RETAINING WALLS, DESIGN. Proportioning of Cantilever Retaining Walls, E. J. Flight. *Concrete and Constr. Eng.*, vol. 28, no. 2, Feb. 1933, pp. 117–125. Theoretical mathematical discussion; numerical examples.

STEEL. Application of Steel Sheet Piling to Engineering Construction—I, S. Packshaw. *Civ. Eng. (Lond)*, vol. 27, no. 318, Dec. 1932, pp. 29–36. II, *Civ. Eng. (Lond.)*, vol. 28, no. 319, Jan. 1933, pp. 16–22.

Practice; development of steel piling; application of permanent work in harbor construction; river protection and sea-defence works; bridge construction; application of high-quality steels; rate of corrosion; sheet piling in tropical conditions; progress of research.

Design; determination of earth pressure; friction between earth and wall; selection of angle of repose; effect of water; design of wall; deflection line method; equivalent beam method; approximate method; design of tie rods and wallings; design of anchorage.

WATER TOWERS, FOUNDATIONS. Tank Piers Carried 50 Ft to Rock Through Old Coal Mine. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, p. 190. Construction of 800,000-gal elevated steel tank, part of the water-supply improvement of Zanesville, Ohio; footings built up to roof of mine support; tall concrete piers reinforced only in upper 25 ft, where soft material was encountered.

HYDRAULIC ENGINEERING

MODELS, TESTING. Slope Table for Fully Controlled Hydraulic Experiments in Open Channels, R. E. Horton. *Water and Water Eng.*, vol. 35, no. 413, Jan. 20, 1933, pp. 5–7. Advantages of control of slope in the testing of steady and unsteady flow of hydraulic models; design of slope table, 120 ft in length, constructed at writer's hydraulic laboratory and used for experiments on movement of flood waves.

HYDRO-ELECTRIC POWER PLANTS

CANADA. Construction Features of Chats Falls Development, H. L. Trotter and J. Dick. *Eng. Inst. Can.—Journal*, vol. 16, no. 2, Feb. 1933, pp. 53–59. Construction of hydro-electric power development on the Ottawa River, having an ultimate capacity of 280,000 hp; building of cofferdams; excavation of 300,000 cu yd of rock; method of handling resulting material; concrete plant placed about 140,000 cu yd; handling of 11,000 tons of equipment and steel; construction of low earth dam, 4,300 ft long, requiring an excavation of 186,000 cu yd.

HYDROLOGY, METEOROLOGY, AND SEISMOGRAPHY

EARTHQUAKE EFFECTS. Effect of Earthquake on Engineering Structures, F. W. Furkert. *Surveyor*, vol. 83, no. 2142, Feb. 10, 1933, p. 205. Effects of two series of severe earthquakes that took place in New Zealand in 1929 and 1931; examples of damage; bridge and road destruction. Before Inst. Civ. Engrs.

RUN-OFF. Run-Off Calculations, A New Method, C. C. Judson. *Inst. Mun. and County Engrs.—Journal*, vol. 59, no. 15, Jan. 17, 1933, pp. 861–867. Outline of method based on the assumption of an ideal run-off, which may tax the sewer system to its utmost.

IRRIGATION

INDIA. Sukkur Barrage Project: Sind (II). *Civ. Eng. (Lond.)*, vol. 27, no. 318, Dec. 1932, pp. 20–25. Building of barrage; underwater work; assembling and placing gates; canal head regulators; operation of gates; drainage; construction of canals; excavating machinery; head regulators of distributaries; criticism of project; cost of project. (Concluded.)

MATERIALS TESTING

IRON AND STEEL, COLD WORKING. Method for Studying Strain Hardening Susceptibility and Aging after Cold Work Deformation, A. Sauveur and J. L. Burns. *Metals and Alloys*, vol. 4, no. 1, Jan. 1933, p. 6. Investigation of Armco iron and S.A.E. carbon steel by Brinell and Rockwell hardness-testing methods.

PORTS AND MARITIME STRUCTURES

BREAKWATERS, COLONIA, URUGUAY. Mole fuer den Freihafen Colonia, Lohrmann. *Bautechnik*, vol. 10, no. 55, Dec. 23, 1932, pp. 743–745. Construction of granite rock breakwater, 720 m long, 6.5 m crown width, 11 m high, in the port of Colonia on the south coast of Uruguay; use of floating cranes; quarrying methods; plan for future extension.

EUROPE. Dock and Harbor Developments, B. Cunningham. *Journal Commerce and Shipping Telegraph—Annual Rev.*, 1932, 11, pp. 63–83. Improvement works at British and western Continental ports.

GREAT BRITAIN. Extension of Southampton Docks, Southern Railway. *Ry. Gas.*, vol. 58, no. 3, Jan. 20, 1933, pp. 79–85. Progress report on the construction of port extension, including the largest graving dock in the world and an ocean quay, capable of berthing simultaneously eight of the greatest ships afloat, or now contemplated.

PIERS, CONSTRUCTION. New Cunard Pier. *Mar. News*, vol. 19, no. 9, Feb. 1933, pp. 45 and 52. Construction of a pier at 14th Street, New York City, finished 10 days ahead of schedule, 8 months after destruction by fire, which occurred May 6, 1932.

PORTS AND HARBORS. Harbors and Waterways in 1932. *Engineer*, vol. 155, no. 4017 and 4018, Jan. 6, 1933, pp. 8 and 9, 1 supp. plate, and Jan. 13, pp. 35–37. Jan. 6: Report on developments in the United Kingdom, including the ports of London, Mersey, and Manchester Ship Canal; northeast coast and Humber ports; Southern Railway ports; ports of South Wales, Scotland, and Ireland; and inland waterways of United Kingdom. Jan. 13: Ports and waterways in Europe, Africa, the East, and America.

SHEET PILING. Carnegie Steel Sheet Piling. Carnegie Steel Company, Subsidiary of United States Steel Corporation, Pittsburgh, Pa., 146 pp. Commercial handbook, containing information of general interest on lateral pressures on walls; design of wharves, slips, retaining walls, and cofferdams; corrosion of steel sheet piling; specifications.

ROADS AND STREETS

ASPHALT. Cut-Back Asphalts—Their Characteristics and Use—I, P. Hubbard. *Roads and Streets*, vol. 76, no. 1, Jan. 1933, pp. 19 and 20. General characteristics of cut-back asphalts; manufacture of cut-back asphalts; value of distillation test for cut-backs. (To be concluded.)

ASPHALT EMULSIONS. Advantages of Emulsified Asphalt in Penetration-Type Construction, V. L. Ostrander. *Can. Engr.*, vol. 64, no. 5, Feb. 7, 1933, pp. 13–17. Differences between emulsion and hot asphalt; characteristics of emulsion and method of application; closing voids; resurfacing old pavements; manipulating unequal depths; covering abandoned rails; varieties of surface finish. Before Asphalt Paving Conference.

ASPHALTIC LIMESTONE. Asphalt Limestone Pavements on Black Base, J. H. Conzelman. *Pub. Works*, vol. 66, no. 1, Jan. 1933, pp. 11 and 39. Construction practice developed by the Alabama State Highway Department in laying,

under contract, a total of over 1,000,000 sq yd of asphaltic limestone pavements.

Roads Commission towards making roads safe for pedestrians; nature of proper footpath; types

tunnel, 10 ft in diameter and 10 miles long, serving the city of Sydney, Australia; bids for construc-

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under contract, a total of over 1,000,000 sq yd of asphaltic limestone pavements.

Bituminous. Beach Sand Builds Bituminous Road, F. W. Haselwood. *Am. City*, vol. 48, no. 2, Feb. 1933, pp. 47–49. Use of beach sand in the resurfacing of 8.4 miles of road at Big Lagoon, Calif.; grading and asphaltic content of surfacing combination; resurfacing done in two 2-in. layers.

Brick. Ohio Tests Brick Pavement Construction Methods, H. H. Wyss. *Pub. Works*, vol. 66, no. 1, Jan. 1933, p. 13. Practice of the Ohio Department of Highways in the construction of 3.063 miles of pavement on State Highway No. 75 in Carroll County, divided into 12 sections, each differing from all others in some respects; features of fillers and cushions used.

Concrete. Boston-Worcester Turnpike Paved by Truck Mixers. *Eng. News-Rec.*, vol. 110, no. 4, Jan. 26, 1933, pp. 104–107. Outline of the practice developed in constructing the new four- and six-lane road from Boston to Worcester, Mass; features of equipment used and exceptional speed processes developed.

Construction. Recent Trends in Surface Treatment, L. P. Street. *Construction Methods*, vol. 15, no. 1, Jan. 1933, pp. 24 and 25. Review of new practices and equipment developed in the United States.

Curves. Solving Vertical Highway Curves by Graph and Table, R. W. Abbett. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, p. 186. Construction of diagram and table for computing points on parabolic vertical curves; single multiplication determines any ordinate from tangent grade to curve.

Highway Systems, Economics. Highway Costs and Who Pays Them, W. Chevalier. *Bus Transportation*, vol. 12, no. 1, Jan. 1933, pp. 22–25. Examples of charts showing unfair and misleading presentation of factual data by railroads.

Highway Traffic Signs, Signals, and Markings. White Cement Traffic Lines. *Construction Methods*, vol. 15, no. 1, Jan. 1933, pp. 36 and 37. New Jersey Highway Commission policy of marking newly constructed routes with inlaid white cement stripes.

Illinois. Future Road Program Planned for Illinois. *Eng. News-Rec.*, vol. 110, no. 8, Feb. 23, 1933, p. 256. Higher standards of engineering administration and accomplishment set forth in report to state legislature by commission on future road program.

Intersections. Safety at Crossroads, T. F. Thomson. *Surveyor*, vol. 83, no. 2141, Feb. 3, 1933, pp. 173 and 174. Treatment of road junctions in undeveloped areas, with regard to safety of average motorist.

Lighting. Necessity of Adequate Highway Lighting, F. T. Groome. *Can. Engr.*, vol. 64, no. 6, Feb. 7, 1933, pp. 19–21. Night accident hazard; economic loss incurred by accidents; dazzling motor headlights.

Long Island. Super-Roads to Serve Pleasure-Land, C. V. Powell. *Eng. News-Rec.*, vol. 110, no. 7, Feb. 16, 1933, pp. 209–212. First of three articles describing a system of superhighways in Queens Borough to provide gateways from New York City to parkways, embracing the beach, sport clubs, and resort areas of Long Island.

Low-Cost. Pennsylvania Applies Spread-Work Principle to Construction of Low-Cost Rural Roads. *Construction Methods*, vol. 15, no. 1, Jan. 1933, pp. 28–31. Methods used in carrying out Pennsylvania's program of rapid construction of 20,000 miles of township roads; organization of camps and unemployment relief.

Macadam. New York State Builds Cement-Bound Macadam Highway. *Pub. Works*, vol. 66, no. 1, Jan. 1933, pp. 9 and 10. Construction of cement-bound pavement 5,026 ft long, 18 ft wide, and 6 in. thick at Newburgh, N.Y.; data on labor, equipment, and costs.

Queens Parkway. Intricate Intersections on Queens Parkway System, C. U. Powell. *Eng. News-Rec.*, vol. 110, no. 8, Feb. 23, 1933, pp. 251–255. Grade separation at all intersecting highways forms keynote to the design of arterial parkways being built in Borough of Queens, New York City; parkway system of 24 miles requires 78 street-crossing bridges and a great variety of route arrangements for the interchange of intersecting traffic; loop designs; complex example.

Road Machinery, United States. Road Equipment Exhibits Unusual Refinement. *Eng. News-Rec.*, vol. 110, no. 4, Jan. 26, 1933, pp. 115 and 116. Review of the exhibits of 121 manufacturers of road building equipment and materials at the Detroit Road Show, held in connection with highway and building congress; equipment less in volume than usual but machines greatly advanced in design and mechanical control; light-weight and abrasion-resisting alloys, welded frames, anti-friction bearings, and improved power units; departmental exhibits.

Sidewalks. Footpaths Along Highways, A. K. Hay. *Contract Rec.*, vol. 47, no. 5, Feb. 1, 1933, pp. 93–95. Work of Ottawa Suburban Roads Commission towards making roads safe for pedestrians; nature of proper footpath; types of sidewalks; practice in vicinity of Ottawa.

United States. State and County Highway Construction in 1932 and 1933. *Roads and Streets*, vol. 76, no. 1, Jan. 1933, pp. 10–17 and 23. Reports from highway officials showing probable expenditures; estimated county road and bridge expenditures for 1933.

SEWERAGE AND SEWAGE DISPOSAL

Activated Sludge. Observations on Bulking in Surface Aeration Activated Sludge Plant, G. M. Ridenour. *Sewage Works Journal*, vol. 5, no. 1, Jan. 1933, pp. 74–82. Relation of sewage purification to amount of solids in aeration tank; characteristic settling curves for different amounts of solids in aerators; operating data and bulking experience at surface aeration; activated sludge plant treating normal domestic sewage. Bibliography.

Assessments. California Sewer-Rental Bill. *West. Construction News and Highways Bldr.*, vol. 8, no. 3, Feb. 10, 1933, pp. 82 and 83. Text of act to provide for acquisition, construction, and operation of sewage works; establishment, and collection of service rates for operating and maintaining such works; also providing for use of such works by other cities, counties, or districts.

Equipment. Mechanical Equipment in Sewage Treatment Works—VIII, A. P. Folwell. *Pub. Works*, vol. 66, no. 1, Jan. 1933, pp. 14–16. Disposing of sludge; sludge beds; sludge handling; vacuum filters. (Continuation of serial.)

Great Britain. Construction Particulars of New Dagenham Sewage Works, T. P. Francis. *Inst. Mun. and County Engrs.—Journal*, vol. 59, no. 15, Jan. 17, 1933, pp. 868–876. Structural details of sludge tanks, Dorr clarifiers, settling tanks, pumping station, and screening chamber.

Odor Control. Control of Odors, Wm. A. Ryan. *Sewage Works Journal*, vol. 5, no. 1, Jan. 1933, pp. 89–92. Summary of methods for the control of odors in sewage disposal plants, with special reference to the experience of Rochester, N.Y. Before Pa. Sewage Works Conference.

Operation. Difficulties in Winter Operation, R. R. Cleland. *Sewage Works Journal*, vol. 5, no. 1, Jan. 1933, pp. 111–116 (discussion) 116. Winter operation of screen chambers, Imhoff tanks, dosing and trickling filters, chlorination equipment, secondary sedimentation basins, sludge drying beds. Bibliography. Before Pa. Sewage Works Ass'n.

Outfall. Repair of Main Outfall Sewer, W. Rolinson. *Inst. Mun. and County Engrs.—Journal*, vol. 59, no. 16, Jan. 31, 1933, pp. 901–907. Method of repairing break in 36-in. cast-iron sewer outfall, 1,210 ft long, at Gosport, Hants, England; repairs cost £800.

Reclamation. Sewage Plant to Sell Effluent to Railroad for Industrial Use, J. E. White. *Mun. Sanitation*, vol. 4, no. 1, Jan. 1933, pp. 16 and 20. City of Herington, Kans., constructs a new sewage disposal plant on the basis of a contract to supply over 300,000 gal of purified effluents daily to shops of the Rock Island Railroad.

Sewers, Design. Determining Stresses in Semi-Elliptical Sewers, C. D. Williams. *Eng. News-Rec.*, vol. 110, no. 8, Feb. 23, 1933, pp. 242 and 243. Analysis of arch ring under trench conditions of loading by tabular method of computation.

Sewers, Maintenance and Repair. Rebuilding Old Sewers Without Interruption of Flow, E. H. Paffrath. *Mun. Sanitation*, vol. 4, no. 1, Jan. 1933, pp. 11–13. Practice of the City of St. Louis in reconstructing old and worn-out sewers, which is being carried on at the present time; flumes installed to direct flow; gunite used for resurfacing; description of original construction; air drying; manholes.

South Africa. Sewage Disposal at Johannesburg, South Africa, E. J. Hamlin and H. Wilson. *Surveyor*, vol. 83, no. 2142, Feb. 10, 1933, pp. 203 and 204. Description of new disposal works; problem of odors; fly breeding; sludge removal; activated sludge plants; elimination of odor during purification.

STRUCTURAL ENGINEERING

Statically Indeterminate Structures, Design. Le calcul des systèmes hyperstatiques au moyen du théorème de Castigliano, R. Alexandre. *Arts & Metiers*, vol. 86, no. 148, Jan. 1933, pp. 13–29. Design of statically indeterminate systems by means of the Castigliano theorem; apparent discrepancies of the Castigliano theorem; application of the Castigliano theorem to the design of fixed and non-fixed systems.

TUNNELS

Concrete Lining. Potts Hill-Waterloo Pressure Tunnel. *Commonwealth Engr.*, vol. 20, no. 6, Jan. 2, 1933, pp. 169 and 170. Failure of reinforced-concrete lining of a circular pressure tunnel, 10 ft in diameter and 10 miles long, serving the city of Sydney, Australia; bids for construction of steel lining.

Construction, Oregon. Owyhee Tunnels, P. R. Hines. *Explosives Engr.*, vol. 10, no. 12, Dec. 1932, pp. 363–370 and vol. 11, no. 1, Jan. 1933, pp. 20–25. Locality map, profile, and sections, and construction procedure; tunnel No. 1 is $3^1/_2$ miles long and 16 ft 7 in. in diameter; tunnel No. 5 is 4 miles long and 9 ft 3 in. in diameter; geological data; survey methods; preparatory work; details of tunnel-driving procedure. Before Am. Inst. Min. and Met. Engrs., Feb. 1933.

Subaqueous. Suspended Tunnel, T. R. Hasley. *Eng. News-Rec.*, vol. 110, no. 6, Feb. 9, 1933, p. 198. Features of scheme for construction of suspended vehicular tunnel, to be held by anchors, under the Straits of Mackinac, Mich.

Water Supply, Massachusetts. Metropolitan District Water Supply Tunnel from Ware River to Wachusett Reservoir, F. E. Winsor. *Boston Soc. Civ. Engrs.—Journal*, vol. 19, no. 9, Nov. 1932, pp. 461–487. Brief history of the Boston Metropolitan District Water Supply; construction of the so-called Quabbin aqueduct tunnel 24.5 miles long, internal diameter of 13 ft; construction of shafts; tunnel unwatering; Ware River intake works; operation of intake works; tunnel bulkhead; tunnel spillway; yield of old and new sources.

WATER RESOURCES.

Underground. Methods for Estimating Ground-Water Supplies, O. E. Meinzer. *Water and Water Eng.*, vol. 35, no. 413, Jan. 20, 1933, pp. 25–28. Safe yield as compared with natural intake and discharge; leakage methods; rock formations as conduits; methods based on laboratory determinations of permeability; method based on permeability determined from discharge and drawdown of wells; method based on area of influence of wells; method based on movements of water levels in relation to rates of withdrawal; evaluation of extraneous influences on water levels.

WATER TREATMENT

Filtration Plants, Specifications. Specifications for Filter Construction, Wm. E. Stanley. *Am. Water Works Ass'n—Journal*, vol. 25, no. 1, Jan. 1933, pp. 105–112. Types of specifications; trend in specifications; filter-plant capacities; major construction items; filter sand and gravel; selection of manufactured equipment; filter-control equipment; chemical feed and sterilization devices; laboratory.

Sacramento, Calif. Water Pre-Treatment Works at Sacramento, D. M. Hoffman. *West. Construction News and Highways Bldr.*, vol. 8, no. 3, Feb. 10, 1933, pp. 75 and 78. History of the water treatment plant of Sacramento since 1852 and description of the new pre-treatment works consisting of 50 by 50-ft Dorrco detritor; blower system; four mixing tanks of heavily reinforced concrete; 56 ft in diameter and 23 ft high; "Spirovortex" pump-mixing units; Dorrco rotary traction-clarifier; Link-Belt "straight-line" sludge-removal units; Fairbanks-Morse 6-in. sludge pump, etc.

Water Analysis. Determination of Total Dissolved Solids in Water Analysis, C. S. Howard. *Indus. and Eng. Chem.* (*Analytical Edition*), vol. 5, no. 1, Jan. 15, 1933, pp. 4–6. Determination of total dissolved solids in carbonate waters, sulfate waters, chloride waters, nitrate waters, and colored waters; drying and ignition of total solids; relation between conductance and total solids. Bibliography.

WATER WORKS ENGINEERING

Management. Water Rates and Construction Policies in Municipally Owned Plants, L. R. Howson. *Am. Water Works Ass'n—Journal*, vol. 25, no. 1, Jan. 1933, pp. 79–86. Construction policy; financial policy; administration.

Pumped Storage. Water-Supply Project, Involving Pumped Storage. *Am. City*, vol. 48, no. 2, Feb. 1933, pp. 60–62. Construction of new Hoopes Dam and pumped storage reservoir for reinforcing the water supply of the city of Wilmington, Del.; the dam is of concrete gravity type, 135 ft high and 900 ft long; details of pumping station at dam; quantities and unit prices; landscaping of area.

Virginia. Municipal Water Supplies of Virginia, F. H. Fish and Others. *Va. Polytechnic Inst.—Bul.*, no. 28, vol. 25, no. 2, Dec. 1932, pp. 7–51. Description of water supplies for incorporated cities and towns, colleges, institutes, private companies, etc.; map showing geographic location of supplies for which analyses are given; chemical analyses.

Wilmington, Del. Double Pumping—to and from a Storage Reservoir, W. C. Wills. *Water Works and Sewerage*, vol. 80, no. 1, Jan. 1933, pp. 32 and 33. Features of Edgar Hoopes storage reservoir, including a concrete gravity dam 135 ft high and 900 ft long; function of reservoir in Wilmington water works.

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